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RECORD OF REVISIONS

Rev	Date	Description	POC	OIC
0	6/28/99	Initial issue (as FEM)	Doug Volkman, *PM-2*	Dennis McLain, *FWO-FE*
1	2/09/04	Changed FEM to ESM; Incorporated IBC & ASCE 7 in place of UBC 97; Incorporated DOE-STD-1020-2002 in place of DOE-STD-1020-94; Incorporated concepts from DOE O 420.1A	Mike Salmon, *FWO-DECS*	Gurinder Grewal, *FWO-DO*

RESPONSIBLE ENGINEERING STANDARDS POC AND COMMITTEE

for upkeep, interpretation, and variance issues

PC-3 and PC-4	**Structural POC/Committee.**

III PC-3 AND PC-4 DESIGN AND ANALYSIS REQUIREMENTS

A. This Part provides requirements for the design and analysis of PC-3 and PC-4 building structures (Subsection III.1.0), nonstructural components, equipment, distribution systems, and non-building structures (Subsection III.2.0). Structural design requirements generally consist of the following:

- Establish structural arrangement/geometry
- Establish loads and load combinations
- Evaluate the structural response to the loads
- Specification of structural capacity and drift limits (acceptance criteria)
- Special design considerations, such as ductile detailing requirements

B. In general, facilities classified as PC-3 or PC-4 have missions that are critical to the National Nuclear Security Administration (NNSA), or contain operations with significant risk potential to public, worker, and environment safety. Following the graded approach philosophy outlined in DOE Order 420.1A, "Facility Safety," and implemented in guidance document, DOE G 420.1-2, and design/evaluation standard, DOE-STD-1020, the design of PC-3 and PC-4 structures, systems, and components (SSCs) follows more stringent and conservative methods than used in model building codes, but more like methods used in practice for nuclear power plant design. Seismic design is accomplished through the use of dynamic analysis methods.

1.0 PC-3 AND PC-4 BUILDING STRUCTURES

1.1 Structural Loads

A. PC-3 and PC-4 buildings are subject to gravity and natural phenomena loads common to all structures. Such loads are defined in model building codes, including the International Building Code (IBC) and ASCE 7. For PC-3 and PC-4 structures, natural phenomena loads are defined based on site-specific studies at specified annual probabilities of exceedance in accordance with DOE-STD-1020. In addition, PC-3 and PC-4 facilities may contain operations of high energy systems (i.e., systems operating at high pressure, temperature, or flow) that can induce structural loads due to normal operations or due to accidents. Loads listed in Table III-1 shall be considered in the design of PC-3 and PC-4 building structures at LANL. Criteria for assessment and mitigation for other natural phenomena loads listed in Appendix C of DOE G 420.1-2, but not addressed in Section III (e.g., volcanic events, lightning, forest fires, drought, fog, frost, extreme temperatures, etc.) must be developed on a site-specific basis per DOE G 4201.-1. LANL FWO-DECS is currently developing design bases for these other loads. Ashfall and earthquake-induced differential movements at the ground surface are loads that need to be considered for PC-3 and PC-4 structures. LANL will provide site specific loads for these phenomena to be considered for the facility structural design.

B. Each of the loads listed in Table III-1 is discussed in more detail in the remainder of this section and minimum loads to be used for design of structures are defined. Wind, snow, and earthquake loads appropriate for LANL are specified herein. In addition, there are LANL specific loads beyond those specified in the building code that are defined in this section.

1.1.1 Dead Load (D)

A. Dead loads consist of the weight of all materials of construction incorporated into the building including walls, floors, roofs, ceilings, built-in partitions, finishes, cladding, and other similar architectural and structural items. Also, fixed service equipment weight including that of cranes, HVAC ducting and equipment, permanent electrical and mechanical equipment and permanent distribution systems comprise dead load. Best estimates of the actual weights comprising the dead load shall be used in design. 10 psf shall be added to the best estimate dead load for all floors for use in design to accommodate future dead load.[1]

Table III-1 Design Structural Loads

Loads from the IBC and ASCE 7	Loads specific for LANL structures
• Dead load • Live load and roof live load • Wind load • Snow load • Rain load • Self straining forces • Load due to fluids • Lateral soil pressure loads • Flood loads • Earthquake loads	• Experimental blast loads • Accidental blast loads • Operating loads from high energy systems • Accident loads from high energy systems

1.1.2 Live Load, (L) and Roof Live Load, (L_r)

1. Live loads (L) are those loads produced by the use and occupancy of a building or other structure and do not include construction or environmental loads as wind load, snow load, rain load, earthquake load, flood load, or dead load. The live load to be used in design shall be determined following the provisions of ASCE 7 [11]. Live load provisions include uniformly distributed loads, concentrated loads, impact loads, reduction in uniformly distributed live loads, and crane operating loads.

B. Live loads used in the design of buildings shall be in no case less than the minimum uniformly distributed loads required in Table III-2. Floors or other similar surfaces shall be designed to support safely the uniformly distributed load or the concentrated load given in Table III-2, whichever produces the greater load effects. Unless otherwise specified, the indicated concentrated load shall be assumed to be uniformly distributed over an area of 2.5 feet square and shall be located so as to produce the maximum load effects in the structural members.

C. Live loads on a roof (L_r) are those produced (1) during maintenance by workers, equipment, and materials; and (2) during the life of the structure by movable objects such as planters and by people. All roofs at LANL shall be designed for a minimum roof live load of 30 psf.[2]

1 The additional 10 psf load is an additional LANL specific requirement that is conservatively applied.
2 The 30 psf minimum roof live load at LANL is specified to cover potential overloads caused by snow, rain on snow, and ashfall, and is based on judgment and history.

1.1.3 Wind Loading (W) and Wind Driven Missiles (W_m)

A. Wind loading (W) shall be calculated using the procedure prescribed in Chapter 6 of ASCE 7 [11]. Wind loads are external and internal pressures acting on all surfaces of the building. External pressures on windward walls are positive in sign signifying pressure acting toward the wall surface. External pressures on leeward walls or the roof are negative in sign signifying pressure acting away from the wall or roof surface (i.e., suction). Internal pressures are considered in partially enclosed or enclosed buildings and must be considered to act both toward and away from the internal surfaces. Wind pressures are determined from the product of the velocity pressure, q_z, a gust effect factor, G, and force (drag) coefficient, C_f. Velocity pressure is a function of the basic wind speed, V, specified for building design.

Table III-2 Minimum Live Load[3]

Use	Uniform, psf	Concentrated, lbs*
Access floor systems		
Office use	50	2000
Computer use	100	2000
Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Lobbies	100	
Movable seats	100	
Platforms (assembly)	100	
Stage floors	150	
Corridors – first floor	100	
Manufacturing and Laboratories		
Light	125	2000
Heavy	250	2000
Office Buildings		
Lobbies and first floor corridors	100	2000
Offices	50	2000
Corridors above first floor	80	2000
Storage Warehouses		
Light	125	
Heavy	250	
Roof (LANL specific minimum requirement)	30	
* Applied uniformly distributed over an area 2.5 feet square		

B. The basic wind speed, V, used for determination of design wind loads on buildings and other structures from DOE-STD-1020 is given in Table III-3 for PC-3 and PC-4. The basic wind speed is defined as a 3 second gust speed at 33 feet above the ground in exposure Category C per ASCE 7.

C. Velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00195 * K_z * K_{zt} * K_d * V^2 * I \ (\mathrm{lb/ft^2; V\ in\ mph}) \qquad \text{Eq. III-1}$$

3 Loads are for typical LANL usage. See Table 4-1 of ASCE 7 for additional minimum loads.

D. Equation III-1 differs from Equation 6-15 of ASCE 7 because of adjustments for the altitude and temperature conditions at LANL. The constants in ASCE 7 reflect the mass density of air for the standard atmosphere, i.e., temperature of 59°F and sea level pressure. The above equation used LANL site conditions from Ref. 33 for elevation of 7380 feet and air temperature of 48.1°F.

Table III-3 Design Wind Speeds

Performance Category	Average Return Period	Basic Wind Speed, V
PC-3	1000 years	117 mph
PC-4	10,000 years	135 mph

E. Disregard shelter from changes in the ground elevation and the height of other buildings by using Exposure C.[4] The importance factor (I) for PC-3 and PC-4 structures for wind loading is 1.0 per DOE-STD-1020 [1].

F. The factor K_{zt} in Equation III-1 is a topographic factor that accounts for wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography. The mesas at LANL that are preferred building sites due to flood considerations may be subject to wind speed-up effects. These effects shall be considered by Section 6.5.7 and Figure 6-4 of ASCE 7.

G. At LANL, tornadoes are not a governing design threat for PC-3 and PC-4 structures as shown in DOE-STD-1020. However, minimum missile criteria are specified to account for objects or debris that could be picked up by straight winds or weak tornadoes. A 2x4 timber plank weighing 15 pounds is the specified missile as shown in Table III-4. Recommended impact speed is 50 mph at the maximum height above the ground shown in Table III-4. This missile will break annealed glass and will perforate sheet metal siding, wood siding up to ¾ inch thick, and form board. The missile could pass through a window or weak exterior wall and cause personnel injury or damage to interior contents of the building. The specified missile will not penetrate typical masonry or concrete walls.

Table III-4 Definition of Wind Driven Missiles

	Load	PC-3	PC-4
Wind Driven Missile	W_m	2x4 timber plank, 15 lb. @ 50 mph (horizontal); maximum height is 30 ft.	2x4 timber plank, 15 lb. @ 50 mph (horizontal); maximum height is 50 ft.

4 Using exposure C is a conservative LANL specific requirement, in that it neglects the potential sheltering from other adjacent structures and trees etc.

1.1.4 Snow Loading (S)

A. Snow loading (S) used for roof load in structure design shall be calculated using the procedure prescribed in Chapter 7 of ASCE 7 [11]. The ground snow load, p_g, for PC-3 and PC-4 structures at LANL is shown in Table III-5. The snow loadings for PC-3 and PC-4 buildings have been derived from a statistical study of 77 years of LANL site-specific data. DOE-STD-1020 [1] indicates an NPH annual probability of exceedance for wind of 1 x 10^{-3} and 1 x 10^{-4} (1000 year and 10,000 year return period) for PC-3 and PC-4 buildings, respectively, which is adopted and is appropriate for snow load. The rain on snow surcharge of 5 psf is not considered for PC-3 and PC-4 structures, in accordance with ASCE 7. The snow load must be factored for unbalanced accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration (drifts) as described in Chapter 7 of ASCE 7. The importance factor (I) for PC-3 and PC-4 structures for snow loading is 1.0 to be consistent with DOE-STD-1020 [1] wind load provisions.

Table III-5 LANL Basic Snow Loads

Performance Category	Average Return Period	Ground Snow Load, p_g
PC-3	1000 years	29 psf
PC-4	10,000 years	41 psf

1.1.5 Rain Loading (R)

A. Rain loading (R) used for roof load in structure design shall be calculated using the procedure prescribed in Chapter 8 of ASCE 7 [11]. Roof drainage systems shall include primary drains or scuppers and secondary (overflow) drains or scuppers. The flow capacity of secondary drainage shall not be less than that of the primary drainage system. Each portion of the roof shall be designed to sustain the load of all rainwater that can accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage at its design flow. The rain load on the undeflected roof in psf is given by:

$$R = 5.2 * (d_s + d_h) \qquad \text{Eq. III-2}$$

where d_s is the depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary system is blocked in inches and d_h is the additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow.

B. Ponding refers to the retention of water due solely to the deflection of relatively flat roofs. Such roofs shall be investigated by structural analysis for the larger of rain or snow loads to ensure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) when rain falls on them or meltwater is created from snow on them. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis.

1.1.6 Self Straining Forces (T)

A. The structural design shall consider self-straining forces arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component material, movement due to differential settlements of foundations, or combinations thereof. Unless specifically addressed through analysis, the effects of self straining forces shall be accommodated by the placement of relief joints, suitable framing systems, or other details.

1.1.7 Fluid Loads (F)

A. Fluid load (F) is the load resulting from the pressure of the fluid. These loads are from fluids with well-defined pressures and maximum height (such as fluids in tanks).

1.1.8 Lateral Soil Pressure Loads (H)

A. Subterranean structural walls shall be designed to resist lateral soil pressure loads (H). These include basement walls, foundation walls, retaining walls, walls of underground vaults for storm or waste water drainage or electrical junctions, and underground tanks. These loads are generally due to static lateral pressure of soil and water. Additional lateral pressure from surface surcharge due to bulk material or to fixed or moving loads shall e added, where applicable. Design lateral soil loads are given in Chapter 5 of ASCE 7 [11] or Section 1610 of the IBC [5] as a function of backfill soil material type. These loads are the minimum design loads unless specified otherwise by a soil investigation report approved by LANL.

B. Active pressure and at-rest soil pressure loads are given in ASCE 7 and the IBC. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top are permitted to be designed for active pressure. Underground tanks having walls that can displace inward under lateral soil pressure loading shall be designed (or, in the case of existing tanks, analyzed) for soil pressure loads which take into account these lateral displacements. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. When a portion or the whole of the adjacent soil is below a free water surface, lateral pressures shall be evaluated based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure. Design lateral pressure shall be increased if soils with expansion potential are present at the site based on criteria provided by a geotechnical expert for the project.

C. Lateral soil pressure loads may also result from earthquake ground shaking (per ASCE 4 [6], Section 3.5.3). Seismic induced soil pressure loads shall be included in earthquake loads (E).

1.1.9 Flood Loads (F_a)

A. Information about LANL regional flooding and runoff analyses for local precipitation at individual LANL sites may be obtained through the LANL Chapter 3 POC. LANL shall re-evaluate this flood issue prior to any major project initiation for potential increases in flood level due to increases in area development and reductions in site drainage paths.

B. In accordance with DOE-STD-1020, PC-3 and PC-4 buildings are to be designed for flood hazards at mean return periods of 10,000 and 100,000 years, respectively. DOE-STD-1020 specifies that both regional flood hazards and local precipitation must be considered. Regional flood hazards arise from river overflow, dam failure, or levee/dike failure. LANL mesa top facilities would typically not be subject to regional flood hazards. However, all sites must be designed for the effects of intense local precipitation.

C. The procedure presented in DOE-STD-1020 for design and evaluation for local precipitation is to develop an initial site drainage system (and roof drainage system) for not less than the 25 year, 6 hour storm. The next step is to perform a hydrological analysis for the site based on the characteristics of the site and the site drainage system to establish the level of local flooding around the facility. For this evaluation, the return period rainfall for the appropriate PC level is used. The design of the site drainage system is addressed in the Civil chapter of the ESM. The hydrological analysis to determine the resulting water levels from local precipitation and the assessment as to whether those water levels are acceptable is also covered in the Civil chapter.

D. The evaluation of the structure for the hydrostatic water loads at the resulting water depths plus any hydrodynamic loads is covered by this chapter (Structural) of the ESM. Flood loads shall include both hydrostatic and hydrodynamic loads. Design loads used for structures located in areas prone to flooding due to either regional flooding or during runoff from local precipitation shall follow the provisions in Chapter 5 of ASCE 7 [11]. Structural systems subjected to flood loads shall be designed, constructed, and anchored to resist flotation, buoyancy forces on basement floors, collapse, and permanent lateral displacement due to action of hydraulic loads associated with the design flood.

1.1.10 Earthquake Effects (E)

A. Response of PC-3 and PC-4 buildings for earthquake ground shaking shall be evaluated using dynamic analysis methods as required by DOE-STD-1020 [1]. ASCE 4 [6] presents four dynamic analysis methods that are acceptable for seismic analysis of LANL structures. It is anticipated that the Response Spectrum Method will be the most frequently used approach at LANL. Time History Methods are also acceptable. By either method, seismic input is defined in terms of site-specific design response spectra termed the Design Basis Earthquake (DBE). DBE response spectra are required in three orthogonal directions (two horizontal and one vertical). For time history analyses, DBE spectrum-compatible time histories are available from the LANL Chapter 5 POC.

B. By DOE-STD-1020 [1], the DBE response spectrum is developed from a probabilistic seismic hazard assessment of the site where mean annual exceedance probabilities for the DBE are specified for each performance category. The DBE spectrum shall be a uniform hazard spectrum such that spectral values at all frequencies are at the specified mean annual exceedance probabilities. The specified mean annual exceedance probabilities and corresponding average return periods are presented below:

- PC-3; mean annual exceedance probability = $4x10^{-4}$ (2,500 year return period)
- PC-4; mean annual exceedance probability = $1x10^{-4}$ (10,000 year return period)

C. A probabilistic seismic hazard assessment has been performed for LANL [28][5]. From this assessment, potential earthquake ground shaking was evaluated at all LANL Technical Areas. Due to similarity of ground motion across LANL, a single DBE was developed for seismic design at all LANL sites. Separate horizontal and vertical DBE have been developed for LANL seismic design. The LANL site-specific DBE horizontal and vertical response spectra shape scaled to peak ground acceleration of 1.0g are shown in Figures III-1 & III-2, respectively. These spectra must be scaled at all frequencies by the PGA at the mean annual exceedance probability specified for the performance category considered. These PGA values are:

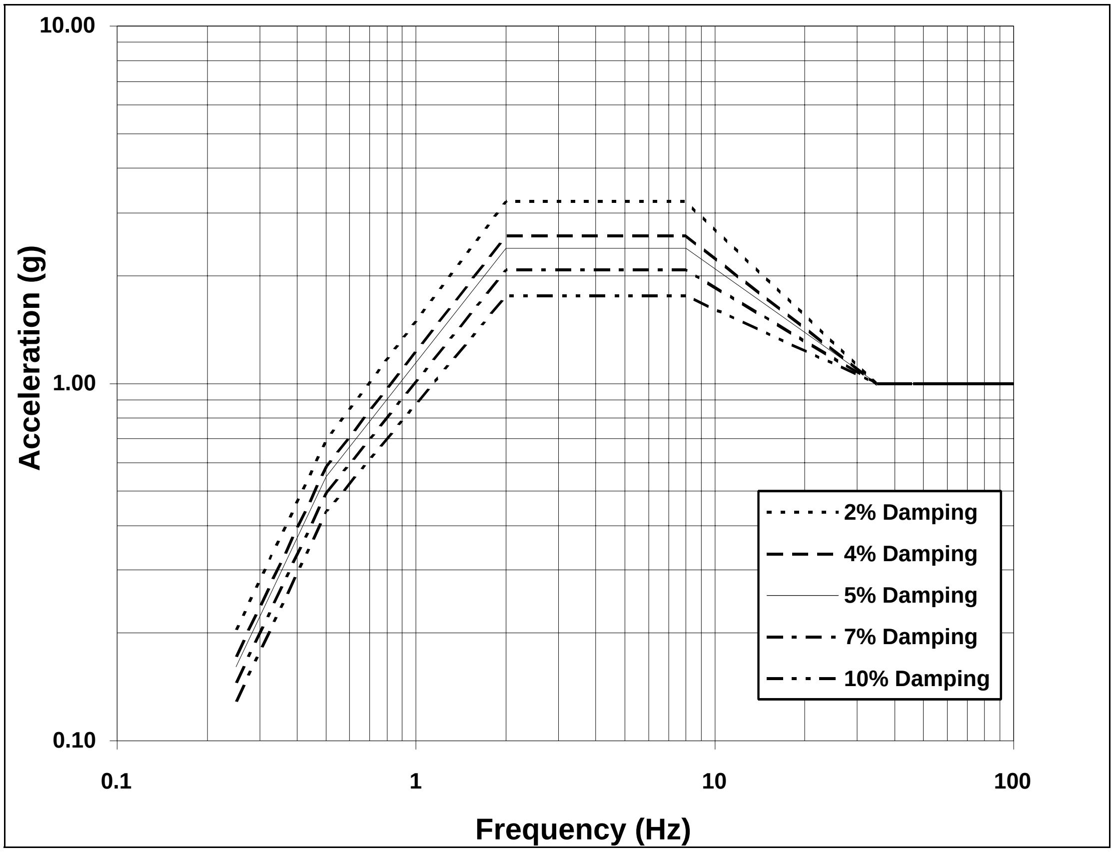
- PC-3; peak ground acceleration (PGA) = 0.34g (2,500 year return period)
- PC-4; peak ground acceleration (PGA) = 0.58g (10,000 year return period)

D. The PGA values presented above are applicable to both horizontal and vertical earthquake motion. Note that the resulting DBE response spectra are for earthquake ground motion at the free ground surface.

E. For time history analyses, one or more recorded, modified recorded, or synthetic earthquake ground motion time histories must be developed. It is anticipated that for the design of LANL structures, linear seismic analyses will be conducted. For such analyses, recorded, modified recorded, or synthetic earthquake ground motion time histories may be used. However, for linear analyses, it is not essential to maintain realistic Fourier phasing as is done with recorded or modified recorded motions. For nonlinear seismic analyses by time history methods, recorded or modified recorded motions shall be used and realistic earthquake Fourier phasing shall be maintained. Earthquake ground motion time-histories in compliance with the requirements presented herein are available from the LANL Chapter 5 POC.

5 LANL should verify that the ground motion associated with the design basis has not been updated prior to releasing the project bid documents. The design engineer should confirm that the design basis earthquake is current since the probabilistic seismic hazard assessment is frequently updated

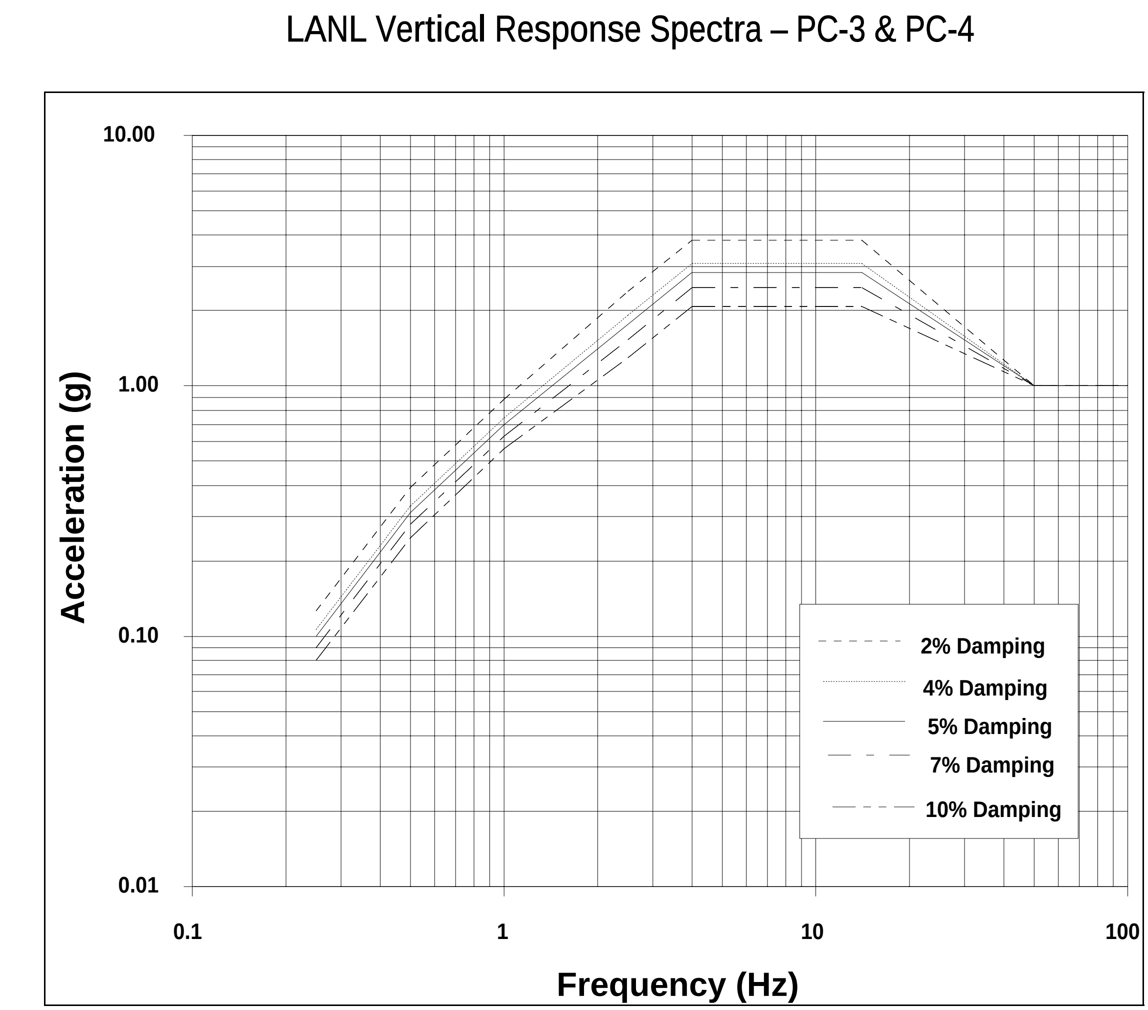
LANL Horizontal Response Spectra – PC-3 & PC-4



Spectral Ordinates for Horizontal DBE

Frequency	2% Damping	4% Damping	5% Damping	7% Damping	10% Damping
0.25	0.20	0.17	0.16	0.15	0.13
0.5	0.69	0.58	0.55	0.49	0.44
2	3.23	2.59	2.39	2.08	1.75
8	3.23	2.59	2.39	2.08	1.75
35	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	1.00	1.00

Figure III-1 DBE Horizontal Response Spectral Shape (Scaled to PGA of 1.0g)



Spectral Ordinates for Vertical DBE

Frequency	2% Damping	4% Damping	5% Damping	7% Damping	10% Damping
0.25	0.13	0.11	0.10	0.09	0.08
0.5	0.39	0.33	0.31	0.28	0.25
1	0.89	0.75	0.70	0.63	0.56
2.5	2.38	1.91	1.76	1.53	1.29
4	3.83	3.07	2.83	2.46	2.08
14	3.83	3.07	2.83	2.46	2.08
50	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	1.00	1.00

Figure III-2 DBE Vertical Response Spectral Shape (Scaled to PGA of 1.0g)

F. Time histories shall be compatible with the DBE response spectrum in accordance with the requirements of ASCE 4 [6]. These requirements are:

- Response spectral values computed from time histories shall be calculated at sufficient frequency points to produce accurate response spectra per Table 2.3-2 of ASCE 4 or, alternately, at frequencies such that each frequency is within 10% of the next lower frequency.
- The mean of the ZPA values calculated from the individual time histories shall equal or exceed the DBE PGA.
- In the frequency range of interest for design of SSCs, the average of the ratios of the mean spectrum (calculated from the individual time history spectra at soil-structure system damping or, conservatively, at the structure damping) to the DBE spectrum, where ratios are calculated frequency by frequency shall be equal to or greater than 1.0.
- No point of the mean spectrum (from the time histories) shall be more than 10% below the DBE spectrum.
- The average of the power spectral densities computed from the individual time histories shall be shown to possess adequate power at all frequencies. In lieu of this ASCE 4 requirement, the computed 5% damped mean spectrum (from the time histories) shall not exceed the DBE spectrum at any frequency by more than 30% in the frequency range between 0.2 and 25 hz. Otherwise, the power spectral densities must be evaluated per Section 2.4 of ASCE 4.

G. In addition, time histories shall have sufficiently small time increment and sufficiently long duration. For seismic analysis of LANL structures, the time increment shall be no larger than 0.010 seconds and the total duration shall be at least 13 seconds including rise time, duration of strong motion, and decay time as defined in Figure 2.3-1 and Table 2.3-1 of ASCE 4. It may be seen from the LANL probabilistic seismic hazard assessment [28] that earthquakes that range in magnitude from 6 to 6.5 are the dominant contributors to the mean seismic hazard. When responses from the three components of motion are calculated simultaneously on a time history basis, the input motions in the three orthogonal directions shall be statistically independent per the requirements in ASCE 4.

1.1.11 Designed Experiment Blast Load (L_{EB})

A. LANL conducts experiments involving explosions. When such experiments take place within a building, the experimental explosion effects shall be contained within an internal structure such that loadings on the building are minimized. The design of the internal containment structure is not within the scope of this chapter. However, the containment structure may impose reaction forces on the building during the experimental explosion. Such reaction forces are one form of designed experiment blast loads, L_{EB}. In addition, experimental explosions may take place exterior to the building under consideration. Blast effects on the building from such exterior experimental explosions are another form of designed experiment blast loads, L_{EB}. The forcing function and duration of the blast loading shall be based on the TNT equivalency of the maximum quantity and closest possible distance from the structural component in question of explosives and propellants in the designed experiment in accordance with TM-5-1300 [22] or reaction forces from the design of the internal containment structure provided by LANL. The dynamic characteristics of these short duration blast loads shall be considered in building evaluation and design. For external explosions, potential fragments and ground shock shall be considered in addition to blast overpressure.

1.1.12 Accidental Blast Load (A_B)

A. Permanent explosives facilities shall comply fully with TM 5-1300 [22] and DOE/TIC-11268 [32]. Blast resistant design for personnel and facility protection shall be based on the TNT equivalency of the maximum quantity of explosives and propellants. In accordance with TM 5-1300 [22], the TNT equivalency shall be increased by 20% for design purposes. *Accidental explosions might also result from the storage and handling of flammable materials, such as hydrocarbons. A release of flammable vapor in a region of adequate confinement and obstacle density is a potential source of a vapor cloud explosion. The blast load resulting from a potential vapor cloud explosion, in terms of incident side-on overpressure and the associated impulse or duration may be estimated using the CCPS book, "Guidelines for Evaluating the Characteristics of Vapor Cloud Explosions, Flash Fires, and BLEVEs" [45].* Blast load effects shall be provided by LANL.

B. The design of all new facilities, or those with major modifications, shall conform to the DOE Explosives Safety Manual, DOE 440.1-1 requirements for either accidental explosions of explosives or vapor cloud explosions. Protective construction design features are provided in TM 5-1300 and DOE/TIC-11268. When evaluating for accidental blast load, the loading A_B shall replace E (earthquake) loads in the load combination equations. All potential blast effects shall be considered including blast overpressure, gas pressure, fragments, and ground shock.

1.1.13 Abnormal Loads Associated with Nuclear Facilities

A. Accidents during operation of high energy systems could lead to loads on structures, systems, and components. The definition and consideration of Design Basis accident loadings is beyond the scope of this document. Facility Specific requirements as determined by the interim or final Safety Analysis Report should be consulted for the applicability of accident type loadings. Design basis accident conditions for the building under consideration will be provided by LANL, if applicable. Examples of accident loads are listed below:

- Accident Maximum Differential Pressure Load (P_a) - This load is due to the maximum differential pressure load build up within a structure by the postulated accident.
- Accident Maximum Differential Temperature Load (T_a) - This load is due to the maximum differential thermal change within a structure by the postulated accident.
- Accident Pipe and Equipment Reactions (R_a) - This load is due to the pipe and equipment reactions generated from the postulated accident.
- Accident High Energy Line Break Reactions (Y_r) - This load is the reaction on the structure generated by the reaction of the broken high-energy pipe during a postulated accident.
- Accident High Energy Jet Impingement Load (Y_j) - This load is the jet impingement load on the structure generated by the steam from a broken high-energy pipe during a postulated accident.
- Accident Missile Impact Load (Y_m) - Missile impact load, such as a pipe whip event generated by or during a postulated accident.

1.2 Load Combinations

A. PC-3 and PC-4 buildings and other structures shall be designed from structural capacities determined by either Strength Design (SD) or Load and Resistance Factor Design (LRFD), for all of the loads described above, except for accidental blast loads. Specific criteria for low probability, high amplitude, very short duration accidental blast loads based on structural response well into the nonlinear range are specified in this Chapter. A single design approach shall be used exclusively for proportioning elements of a particular construction material throughout the structure.

B. *PC-3 and PC-4 building structures are typically nuclear facilities with potential radiological release consequences. DOE-STD-1020 [1] specifies specific load combinations for earthquake and wind effects on PC-3 and PC-4 structures. There are material codes specifically written for nuclear application, including ACI 349 for reinforced concrete structures and AISC N690 for steel structures, each with their own set of load combination rules. AISC N690 is a variation of the use of Allowable Stress Design in which the allowable capacities are scaled up to approach strength design limits. Since Allowable Stress Design is not permitted for PC-1 and PC-2 steel structures as LANL, it is logical to not permit this approach for PC-3 and PC-4.* AISC LRFD shall be the design code for PC-3 and PC-4steel structures with the load combination modifications specified in DOE-STD-1020 and this document for earthquake and wind effects. Similarly, ACI 349 shall be the design code for PC-3 and PC-4 concrete structures with the load combination modifications specified in DOE-STD-1020 and this document for earthquake and wind effects. For other loads, the load combinations from ASCE 7 [11] are to be used with AISC LRFD or ACI 349 provisions. As a result, load combinations are presented herein in three subsections: (1) loads not including earthquake and wind effects; (2) combinations including earthquake effects; and (3) combinations including wind effects. The load combinations in the first section are adopted from both ACI 349 and ASCE 7. The load combinations for earthquake and wind effects are adopted from DOE-STD-1020.

1.2.1 Combinations Not Including Wind or Earthquake Effects

A. Structures, components and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the combinations presented in Table III-6. Each limit state shall be investigated with the effects of one or more loads not acting (except for Dead Load) when they possibly counteract each other. The load factors and combinations reflect the structural demand uncertainty from the individual loads and their combined effect to potential structural failure regardless of the structural material used in the design. Differences in material reliability are accounted for while determining the structural capacity with capacity reduction factors unique to the particular material and stress state.

B. *The blast loadings that are considered at LANL are not included in basic load combinations. Therefore, the load combinations were appropriately modified to include the blast loadings. Engineered blast load, L_{EB} will most likely affect only limited portions of the building when experiments are conducted. Because these loads can be repeated many times during the life of the structure, the structure should be designed to remain elastic.*

Table III-6 Load Combinations for Strength Design or LRFD
(Not Including Earthquake and Wind Effects)

1.	$1.4 (D + F)$
2.	$1.2 (D + F + T) + 1.6 (L + H) + 0.5 (L_r \text{ or } S \text{ or } R)$
2a	$1.2 (D + F + T) + 1.6 (L + H) + 0.5 L_r + L_{EB}$
3.	$1.2 (D + F) + (1.6 L_r \text{ or } 1.6 S \text{ or } 1.6 R) + (0.5 L \text{ or } 0.8W)$
3a	$1.2 (D + F) + 1.6 L_r + 0.5 L + L_{EB}$
4.	$D + F + L + H + T_a + R_a + 1.25 P_a$
Notes:	
a)	H are normally acting earth pressure loads (not due to earthquake)
b)	In locations of the building where engineered blast loads must be considered for structural design, Load Combinations 2a and 3a shall also be considered.

1.2.2 Combinations Including Earthquake Effects

A. PC-3 and PC-4 structures shall be proportioned and detailed to resist the effects of gravity loads, operating loads, and applicable accident loads along with earthquake loads. Seismic load combinations shall be based on strength based acceptance criteria and shall consider the inelastic energy absorption factor, F_μ and scale factor, SF.

B. For earthquake and other loads, where stress based criteria are employed, the following load combination shall be used:

$$D = D_{NS} + E$$

$$E = \frac{SF * D_S}{F_\mu} \qquad \text{Eq. III-3}$$

where:

- D = the total demand acting on an element.
- D_{NS} = the non-seismic demand acting on an element. Non-seismic demand shall include the mean effects of dead, live, equipment, fluid, snow, and at-rest lateral soil loads as well as accident loads occurring due to the earthquake.
- E = scaled inelastic earthquake effects
- D_S = the calculated seismic response to the DBE using an elastic analysis approach (by either response spectrum or time history analysis)
- F_μ = an inelastic energy absorption factor for structural elements.
- SF = a scale factor based on performance category target performance goals

C. Scale factors are 0.9 for PC-3 and 1.25 for PC-4 as specified in DOE-STD-1020. Alternately, scale factors computed from the slope of the site-specific seismic hazard curves as described in Appendix C of DOE-STD-1020 may be used. The alternate scale factors result in improved achievement of target performance goals.

D. The inelastic energy absorption factor, F_μ, is a reduction factor used to reduce seismic demand to account for limited inelastic behavior. The inelastic energy absorption factor is a function of the structural system as specified in Table III-7. F_μ values given in Table III-7 assume good seismic detailing practice for concrete per ACI 349 [20], and for steel per AISC "Seismic Provisions for Structural Steel Buildings" [18] and IBC 2003. The F_μ values further assume that the inelastic demand is distributed throughout the lateral force resisting system and that the inelastic deformations are not concentrated in a single element or story. In situations where LANL, by specification or from the safety analysis, has limited the structure stress levels to elastic limits, F_μ shall be taken as 1.0.[6]

E. The F_μ values in Table III-7 are not only for structural systems but they are also for specific response modes. The values in Table III-7 are for bending moment, in-plane shear, and axial load in diagonal bracing. For other axial loads, other shear loads, and torsion, F_μ shall be taken as 1.0. Many elements have combined response modes. An example is a column subjected to combined axial force and bending. In this case, the F_μ factor is only applied to moment generally in the following form:

$$\frac{P_{NS} + P_S}{P_{CR}} + \frac{M_{NS} + M_S / F_\mu}{M_P} \leq 1.0 \qquad \text{Eq. III-4}$$

where P_{NS} and M_{NS} are the non-seismic concurrent axial force and moment demand, respectively; P_S and M_S are the seismic axial force and moment demand, respectively; and P_{CR} and M_P are the axial force and moment capacity, respectively.

F. There is a limitation in DOE-STD-1020, that to determine the response of structures where F_μ is greater than unity, the maximum spectral acceleration shall be used for the fundamental mode when the fundamental frequency of that mode is greater than the frequency at which the maximum spectral amplification occurs (i.e., 8 hz as shown in Figures III-1 and III-2). For higher modes, the actual spectral accelerations shall be used per DOE-STD-1020. For LANL application, an alternate method is specified to determine the F_μ value when the fundamental mode is greater than 8 hz. In this case, F_μ shall be determined by:

$$F_\mu = F_\mu^{'} * \left(\frac{SA_f}{SA_{peak}} \right) \geq 1.0 \qquad \text{Eq. III-5}$$

where:

F_μ = the factor to be used in Equation III-3
F'_μ = the factor determined from Table III-7 for the structural system being designed.
SA_f = the spectral acceleration at the frequency of the predominant building mode
SA_{peak} = the peak spectral acceleration

6 The F_μ levels are related to the ability of the structural walls to provide confinement associated with the qualitative definition of NPH PC3 and PC4. Where stricter confinement of gas and airborne particles is needed the F_μ used for design should taken to be 1.0

Table III-7 Inelastic Energy Absorption Factors, F'_μ

Structural System	F'_μ
Moment Resisting Frame Systems – Beams	
Steel Special Moment Resisting Frame (SMRF)	3.0
Concrete SMRF	2.75
Concrete Intermediate Moment Frame (IMRF)	1.5
Steel Ordinary Moment Resting Frame	1.5
Concrete Ordinary Moment Resting Frame	1.25
Shear Walls	
Concrete or Masonry Walls	
In-plane Flexure	1.75
In-plane Shear	1.5
Out-of-plane Flexure	1.75
Out-of-plane Shear	1.0
Plywood Walls	1.75
Dual System, Concrete with SMRF	2.5
Dual System, Concrete with Concrete IMRF	2.0
Dual System, Masonry with SMRF	1.5
Dual System, Masonry with Concrete IMRF	1.4
Steel Eccentric Braced Frames (EBF)	
Beams and Diagonal Braces	2.75
Beams and Diagonal Braces, Dual System with Steel SMRF	3.0
Concentric Braced Frames	
Steel Beams	2.0
Steel Diagonal Braces	1.75
Steel Chevron and Vee Braces	1.5
Concrete Beams	1.75
Concrete Diagonal Braces	1.5
Wood Trusses	1.75
Beams and Diagonal Braces, Dual Systems	
Steel with Steel SMRF	2.75
Concrete with Concrete SMRF	2.0
Concrete with Concrete IMRF	1.4
All Columns	
Flexure	1.5
Axial compression and shear	1.0
Connections*	1.0

* Connections shall be designed for loads corresponding to forces in the connecting members that correspond to member F_μ of unity.

G. The total demand determined from Equation III-3 is to be compared with member capacities in accordance with the applicable material code, as discussed later in this Part.

1.2.3 Combinations Including Wind Effects

A. PC-3 and PC-4 structures, components and foundations subjected to extreme wind loads shall be designed so that their design strength equals or exceeds the effects of the factored loads in the combinations presented in Table III-8. Each limit state shall be investigated with the effects of one or more loads not acting (except for Dead Load) when they possibly counteract each other. The load combinations given in Table III-8 are taken from ASCE 7 with modifications for blast and flood and reduced by 10 percent in accordance with the wind provisions of ASCE 7. Note that design by ASD is permitted for wind design by DOE-STD-1020. It is recommended that strength design or LRFD be used for LANL structures.

Table III-8 Load Combinations for Wind Effects by Strength Design or LRFD

W1.	0.9{1.2 (D + F) + (1.6 L_r or 1.6 S or 1.6 R) + [(0.5 L and/or 0.5 L_{EB}) or 0.8W]}
W2.	0.9 {1.2 D + 1.6W + 0.5 L + 0.5 (L_r or S or R)}
W2a	0.9 {1.2 D + (0.8W + 1.0F_a) + 0.5 L + 0.5 (L_r or S or R)}
W3.	0.9 {0.9 D + 1.6 W + 1.6 H}
W3a	0.9 {0.9 D + (0.8W + 1.0F_a) + 1.6 H}
Notes:	
a)	H are normally acting earth pressure loads (not due to earthquake)
b)	Load factor for L for these Combinations W1 and W2 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where L is greater than 100 psf.
c)	The load factor on H shall be set equal to zero in Combinations W3 and W3a if the structural action due to H counteracts that due to W. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.
d)	In areas where flood loads must be considered for structural design, Load Combinations W2a and W3a shall also be considered.

B. Where flood loads need to be considered in structural design, addition Load Combinations 2a and 3a where 1.6W in Combinations 2 and 3 is replaced by 0.8 W + 1.0 F_a shall be considered. The most unfavorable effects from both wind and earthquake shall be investigated, where appropriate, but they shall not be considered to act simultaneously.

C. Wind generated missiles striking a building produce global response of the building to the impact force and local response of the structural element struck, including perforation, penetration, and scabbing. For the 2x4 missile specified for PC-3 and PC-4 structures, the global response will be trivial compared to seismic or wind loads on the structure. For the case of local missile impact on structures, concurrent loads from other sources are typically not considered.

1.2.4 Combinations Including Accidental Blast Effects

A. PC-3 and PC-4 structures shall be proportioned and detailed to resist the effects of gravity loads, operating loads, and applicable accident loads along with loads from accidental explosions. Accidental blast load combinations shall be based on deformation based acceptance criteria and shall consider inelastic energy absorption by limiting deformations to ductility or plastic hinge rotation limits.

B. Accidental blast loads, A_B, are infrequent events. These events do not have the same level of recurrence data as is available for natural phenomena hazards. Site specific data has not been evaluated. Therefore, in lieu of quantitative information, it is judged that the frequency of occurrence of accidental blasts is of about the same order as PC-3/PC-4 earthquakes. Therefore, load combinations and load factors are specified as follows:

$$D = D_{NB} + F_{AB} D_{AB} \qquad \text{Eq. III-6}$$

where:

D = the total demand acting on an element.
D_{NB} = the non-blast demand acting on an element. Non-blast demand shall include the mean effects of dead, live, equipment, fluid, snow, and at-rest lateral soil loads.
D_{AB} = the calculated blast demand.
F_{AB} = the load factor appropriate to account for the frequency of occurrence of accidental blast loads (1.0 unless data exist)

C. This load factor above, F_{AB}, may be modified to account for the annual frequency of occurrence of accidental blast loads where quantitative data are available. The load factors should be developed considering the NPH performance category for the component being designed. The total demand determined from Equation III-6 is to be compared with permissible inelastic deformations given in terms of ductility limits or plastic hinge support rotation limits. These limits may be found in numerous publications including TM 5-1300 [22] and DOE/TIC-11268 [32]. Deformation limits are a function of the structural system, the mode of response, and the level of acceptable damage.

1.3 Structural Design

A. Buildings shall be designed to support safely the loads in load combinations defined above without exceeding the appropriate strength limits for the given materials of construction. In addition, structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift for comfort of building occupants, for serviceability of the building, and to limit structural damage in the case of occasional loads such as earthquake. Buildings shall be designed in accordance with the applicable material standards (ACI 349 or AISC LRFD).

B. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

C. PC-3 and PC-4 buildings shall be evaluated by seismic dynamic analysis using any of the methods presented in ASCE 4 [6]. The total lateral force due to earthquake, wind, or blast pressure shall be distributed to the various vertical elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Provisions shall be made for increased forces induced on resisting elements of the structural system due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force resisting system. Every structure shall be designed to resist the overturning and/or sliding effects caused by lateral forces from wind, lateral soil loads, earthquake, and blast pressure.

D. Structural design is an iterative process that begins with a conceptual or preliminary design of the building. The building layout and materials of construction are developed to meet functional and cost requirements. For seismic design of building structures, certain lateral force resisting systems and materials of construction are not permitted for PC-3 and PC-4 LANL structures. Also, a certain level of structural detailing is required for PC-3 and PC-4 LANL structures. The acceptability of lateral force systems and the required level of detailing for these LANL structures shall follow ASCE 7 provisions for Seismic Design Category D. The basic seismic force resisting system and its level of detailing must be selected from Table 9.5.2.2 of ASCE 7 as one of the initial steps in the seismic design process. From the preliminary building design, the structural design is completed in the following steps:

   1. Evaluate expected loads on the structure as described in Section III.1.1 of this chapter.
   2. Develop a mathematical model of the structure in order to evaluate structural response to the applied loads. Modeling is described in Section III.1.4 of this chapter.
   3. Perform structural response analysis to determine the load effects in structural members. Load effects are forces, stresses, and deformations resulting from the applied loads. Structural analyses are described in Section III.1.5 of this chapter. For most loads, linear elastic static response analyses are performed. Dynamic analyses and considerations of inelastic response must be addressed for seismic and blast loads.
   4. Determine the maximum response of structural elements by utilizing the load combinations described in Section III.1.2 of this chapter.
   5. Compare the maximum responses to acceptance criteria found in the applicable material standards to assure that the general structural design requirements specified earlier in this section (III.1.3) are met. Assure that stresses and deformations are within acceptable limits. If stresses or deformations are unacceptable, the design must be modified and the process repeated until response stresses and deformations are acceptable. In addition, apply design measures to assure that a reliable and ductile design is achieved. Design acceptance criteria and other design measures are described in Section III.1.6 of this chapter.

1.4 Structural Modeling

1.4.1 General Requirements

A. Structural response to the loads described in Section III.1.1 shall be determined by analysis of a mathematical model of the building structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure. The model shall also represent the spatial distribution of the stiffness and mass (for seismic analyses) throughout the structure. For seismic analysis of PC-3 and PC-4 building structures, the model needs to be capable of representing seismic response in three orthogonal directions, two horizontal and the vertical direction.

B. The model shall include all structural members of the building that experience significant effects (i.e., forces, stresses, and deformation) to the applied loads. The structural model must include the complete load path for vertical loads from the point of application to the foundation and supporting media to properly consider gravity loads. The model must also include the complete lateral load path to properly consider earthquake and wind loads.

C. The amount of detail used to represent a structure in a mathematical model depends on the structural configuration and the use of the model. Finite element mathematical models are used to represent complex structures. A detailed model of the building structure shall be prepared to evaluate the structural response to static loads such as dead load, live load, rain and snow loads, etc. For PC-3 and PC-4 structural design, seismic dynamic analysis is required and the same detailed model may be used to evaluate seismic inertial loads and the structural response to earthquake input motion. Alternately, a more simple model may be prepared which captures dynamic behavior and evaluates seismic inertial loads. For the latter case, the seismic inertial forces may then be applied to the detailed structural model to determine the seismic response in the elements of the structure. A more simple lumped-mass stick model may be used under the following conditions:

   - The horizontal response analysis does not include direct determination of seismic stresses, and the floor slabs can be considered rigid in-plane reducing the total number of dynamic degrees of freedom.
   - The vertical response analysis determines seismic motions at different elevations of the structure, but not the response of various points on the floor. Vertical response may be evaluated using simple bounding models of the floor at each level.

D. The model shall accurately model the stiffness of the structure and parts of the structure such that reasonable response deformations may be computed. Accurate representation of the stiffness also enables reasonable estimation of building dynamic characteristics for dynamic seismic analyses. For dynamic seismic analyses, the model shall accurately model the amplitude and location of mass throughout the structures. Accurate representation of mass enables reasonable estimation of building dynamic characteristics and seismic induced inertial loads for dynamic seismic analyses. The model shall represent the actual locations of centers of masses and centers of rigidity, thus accounting for torsional effects produced by lateral loads and their eccentricity.

E. When calculating forces in various structural elements, the torsional moments due to accidental eccentricity with respect to the center of rigidity and non-vertically incident or incoherent seismic waves shall be accounted for. An acceptable means of accounting for these torsional moments is to include an additional torsional moment in the design and evaluation of structural members. This additional torsional moment shall be taken equal to the story shear at the elevation and in the direction of interest times a moment arm equal to 5% of the building plan dimension perpendicular to the direction of lateral force in the analysis. Consideration of such eccentricity shall be used only to increase the magnitude of the forces. The structural model used for seismic analysis shall not be changed to include accidental torsion.

1.4.2 Modeling of Stiffness

A. Mathematical models shall seek best estimate stiffness properties for structural elements such that reasonable deformations are computed and best estimate dynamic characteristics are achieved. The model shall comply with the following.

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked section.
2. For steel moment resisting frame systems, the contribution of panel zone deformations to overall story drift shall be included.

B. In general, the distribution of loads in structural elements throughout concrete and masonry structures is approximately the same whether gross "uncracked" properties or "cracked" properties are used. However, displacement will be underestimated when uncracked properties are used and there is substantial cracking in the structure. Hence, cracked section properties for concrete or masonry members in the model shall be used when such cracking is projected.

C. Cracking in reinforced concrete and masonry structures are complex phenomena generally making detailed analytical determination of the appropriate properties impractical. Good engineering judgment and experience must play a major role in considering the effect of cracking on the stiffness of concrete. When the effects of cracking are considered, section properties developed based on the maximum internal moments from the various load combinations can be estimated using Eqn. 9-8 in ACI 318 [16] as presented below as Equation III-7.

$$I_E = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \text{Eq. III-7a}$$

where:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{Eq. III-7b}$$

and for normal weight concrete:

$$f_r = 7.5\sqrt{f_c^{'}} \qquad \text{Eq. III-7c}$$

where M_a is the maximum moment in the member, M_{cr} is the cracking moment, I_g is the gross moment of inertia, I_{cr} is the moment of inertia of the cracked transformed section, f_r is modulus of rupture, and y_t is distance from centroidal axis to extreme fiber in tension.

D. When buildings are subjected to earthquakes, limited inelastic behavior is permitted. Such inelastic behavior will result in significant cracking in reinforced concrete and reinforced masonry structures. Hence for elastic seismic analysis, the effective stiffness of reinforced concrete (or reinforced masonry) members provided in Table III-9 shall be used. When finite element methods are used, the element stiffness shall be modified using the effective stiffness factor for the dominant response parameter.

Table III-9 Effective Stiffness of Reinforced Concrete Members

Member	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams – nonprestressed	$0.5E_cI_g$	G_cA_W	
Beams – prestressed	E_cI_g	G_cA_W	
Columns in compression	$0.7E_cI_g$	G_cA_W	E_cA_g
Columns in tension	$0.5E_cI_g$	G_cA_W	$E_s A_s$
Walls and Diaphrams – uncracked , $f_b<f_{cr}$, $V<V_c$	$0.8E_cI_g$	$0.8G_cA_W$	E_cA_g
Walls and Diaphrams – cracked, $f_b>f_{cr}$, $V>V_c$	$0.5E_cI_g$	$0.5G_cA_W$	E_cA_g
E_c = concrete compressive modulus G_c = concrete shear modulus = $0.4E_c$ E_s = steel modulus I_g = gross moment of inertia	A_w = web area A_g = gross area of the concrete section A_s = gross area of the reinforcing steel f_b = bending stress		f_{cr} = cracking stress V = wall shear V_c = nominal concrete shear capacity

E. Materials properties of interest for the purposes of modeling structure stiffness are the modulus of elasticity and Poisson's ratio. These material properties for common structural materials may be taken as follows:

Concrete:	Calculate per Section 3.1.2.1.1 of ASCE 4 [6].
Steel:	Use values given in Section 3.1.2.1.2 of ASCE 4 [6].
Masonry:	Calculate per Section 1.8.2.2.1 of ACI 530 [17].
Aluminum:	Calculate per Section 3.1.2.1.3 of ASCE 4 [6].

F. The type of finite element required to model a structural system shall depend on the type of response desired. The selection of the finite element type shall also consider the analytical theory on which the element is based, in order to represent major characteristics of the structural system. The selection of discretization parameters shall consider the size, shape, and aspect ratio of the elements; the internal node points; and the number of nodes required to define the element. The finite element model shall produce responses that are not significantly affected by further refinement in the element mesh size and shape. The latter requirement may be satisfied based on comparable past experience in lieu of multiple analyses using successively refined mesh.

1.4.3 Modeling of Mass

A. For seismic dynamic analyses, the total mass of the structure and the distribution of mass throughout the structure must be modeled. The total seismic weight of the structure shall include the dead load plus the other loads listed below:

1. A minimum of 25% of the floor live load.
2. Where an allowance for partition load is included in the floor design, the actual partition weight or a minimum weight of 10 psf.
3. Total operating weight of permanent equipment.

B. The inertial properties of a structure may be modeled by assuming that the structural mass and associated rotational inertia are discretized and lumped at node points. Vertical inertia effects need not be modeled for PC-1 and PC-2 structures. The following mass modeling conditions shall be met:

1. Structural mass shall be lumped so that the total mass, as well as the center of gravity, is preserved, both for the total structure and for any of its major components that respond in the direction of motion.
2. The number of dynamic degrees of freedom, and hence the number of lumped masses, shall be selected so that all significant vibration modes of the structure can be evaluated. For a structure with distributed mass, the number of degrees of freedom in a given direction shall be equal to at least twice the number of significant modes in that direction.

C. *The effect and location of the buildings mass can be included in one of two ways; 1) masses may be lumped at nodes by the structural analyst in a manner in which the total mass and c.g. of the structure and major components is preserved, or 2) the structural mass may be modeled using the mass density terms in the structural software used such that lumped masses are automatically generated. In the latter case, the mass from fixed equipment, partitions, and an appropriate percentage of the live load as described above must be included manually. It is generally recommended that masses be manually lumped at nodes such that the analyst best understands the structure dynamic behavior and extraneous modes are not generated that compromise numerical accuracy.*

1.4.4 Modeling of Structural Damping

A. Damping values to be used in linear elastic analyses for determining seismic design loads for structures and for generation of in-structure response spectra are presented in Table III-10 as a function of the average response level in the seismic load resisting elements represented by the demand to capacity ratio, D_e/C [C = code capacity; D_e = total elastic demand ($D_e = D_{NS} + SF*D_S$), including non-seismic loads]. The relationship between D_e/C and response level is:

- $D_e/C < 0.5$ Response Level 1
- $D_e/C \approx 0.5$ to 1.0 Response Level 2
- $D_e/C > 1.0$ Response Level 3

Table III-10 Specified Damping Values for Seismic Response Evaluation

Type of Component	Damping (% of critical)		
	Response Level 1	Response Level 2	Response Level 3
Welded and friction bolted metal structures	2	4	7
Bearing-bolted metal structures	4	7	10
Prestressed concrete structures (without complete loss of prestress)	2	5	7
Reinforced concrete structures	4	7	10
Masonry shear walls	4	7	12

B. *When checking the response of the structure to the design basis earthquake, the use of Response Level 3 damping is recommended for LANL structures. This is an exception to the current version of DOE-STD-1020 [1]. DOE-STD-1020 provides damping values at three response levels as shown in Table III-10. DOE-STD-1020 limits the damping to Response Level 2 values for seismic structural design. However, the F_μ values listed in Table III-7 assume a level of inelastic response commensurate with Response Level 3 damping values. Therefore, unless a reduced F_μ value is used, the Response Level 3 damping values are appropriate. Response Level 2 damping values should be used in those cases where the structure is anticipated to remain elastic when subjected to the design basis earthquake (i.e,. $F_\mu = 1.0$). It is anticipated that future revisions of DOE-STD-1020 will permit the use of Response Level 3 damping values.*

C. Response Level 3 damping may be used for evaluating seismic-induced forces and moments in structural members by elastic analysis without consideration of the actual response. An exception to this requirement is that Response Level 1 damping must be used if elastic buckling conditions in columns control the design.[7] If a nonlinear inelastic response analysis which explicitly incorporates the hysteretic energy dissipation is performed, Response Level 1 damping values shall be used to avoid the double counting of the hysteretic energy dissipation which would result from the use of higher damping values. Response Level 2 damping values may be used if they can be justified.

7 This condition occurs when column D/C ratios are the highest of the members in the lateral load path, and the axial compression stress component contributes more than 60% of the combined column D/C ratio.

D. Consideration of actual response level is required for generation of in-structure response spectra. No greater than Response Level 2 damping shall be used for in-structure response spectra generation. In lieu of iterative analyses to determine the actual response level and associated damping value, Response Level 1 damping values may be used for generation of in-structure response spectra.

E. Modeling of damping as presented in ASCE 4 [6] (Section 3.1.5) shall be followed. Use the composite damping methods given for structural systems that consist of substructures with different damping properties. Alternately, it is conservative to use the lowest damping ratio associated with the material/construction used.

1.4.5 Soil-Structure Interaction (SSI)

A. Soil-Structure Interaction (SSI) effects shall be considered for all LANL PC-3 and PC-4 structures. SSI analysis shall be performed in accordance with the requirements of ASCE 4 [6], Section 3.3. Per ASCE 4, acceptable methods of analysis include the direct method (Section 3.3.3 of ASCE 4) and the impedance-function approach (Section 3.3.4 of ASCE 4). *Additional guidance for performing soil-structure interaction analysis is provided in Section C.4.3 of DOE-STD-1020 [1].* The foundation motion of a structure founded on a soil site during an earthquake can be different than the free field earthquake ground motion provided the structure is sufficiently massive and the soil is sufficiently flexible. When this phenomenon occurs, SSI is significant for that structure and the underlying soil. SSI is not important and a fixed base analysis may be used per ASCE 4 [6], when the dominant frequency of response obtained assuming a rigid structure supported on soil springs is more than two times the dominant frequency of the flexible structural model with a fixed base. ASCE 4 provides guidance on the development of soil spring properties.

B. When massive stiff structures are founded on or embedded in a soil foundation media, both the frequency and amplitude of response due to seismic excitation can be affected by SSI. Various aspects of SSI result in reduced motion of the foundation of a structure from that recorded by instruments in the free field. SSI also results in a shift of the fundamental frequency of the structure. Such a shift can either reduce or increase the response of the structure. The frequency shift resulting from SSI shall be accounted for in seismic analyses for all LANL PC-3 and PC-4 structures where SSI effects are significant. The following general requirements shall be considered when a detailed SSI analysis is performed:

(a) Spatial Variation of Free-Field Motion: Apply Section 3.3.1.2 of ASCE 4 [6].
(b) Three Dimensional Effects: Apply Section 3.3.1.3 of ASCE 4.
(c) Non-linear Behavior of Soil: Apply Section 3.3.1.4 of ASCE 4.
(d) Structure-to-Structure Interaction: Apply Section 3.3.1.5 of ASCE 4.
(e) Effect of Mat and Lateral Wall Flexibility: Apply Section 3.3.1.6 of ASCE 4.
(f) Uncertainties in SSI Analysis: Apply Section 3.3.1.7 of ASCE 4.
(g) Model of structure: Apply Section 3.3.1.8 of ASCE 4. Simplified models may be used as described in Section III.1.4.5.C below.
(h) Embedment Effects: Apply Section 3.3.1.9 of ASCE 4.
(i) Wave Incoherence: Apply Section 3.3.1.10 and Table 3.3-2 of ASCE 4.

C. The DBE response spectra defined in Section III.1.1.10 are free field input at the ground surface. The SSI analysis shall appropriately apply the DBE response spectra or corresponding earthquake time histories.

D. Prior to seismic design of buildings and where SSI effects are determined to be significant, site response analyses shall be performed to determine the best estimate, lower bound, and upper bound soil properties appropriate for earthquake-induced soil shear strains and the free field ground motion at the ground surface. Site response and SSI analyses require the following geotechnical data input:

- low-strain shear modulus (or low-strain shear wave velocity),
- low-strain elastic modulus (or low-strain compression wave velocity),
- Poisson's ratio,
- unit weight,
- degradation curves indicating the variation of shear modulus with shear strain,
- low strain soil material damping,
- degradation curve indicating the variation of soil material damping with shear strain, and the elevation beneath the structure where rock is encountered.

E. Subsurface material properties shall be determined by field and laboratory testing, supplemented as appropriate by experience, empirical relationships and published data for similar materials as described in Section 3.3.2 of ASCE 4 [6]. A determination must be made as to how soil properties vary with depth from the free ground surface to well below the structure foundation. This is required for both the SSI and site response analyses. *In particular, the variation with depth down to the underlying basalt at LANL may be important for the site response analysis.*

F. There is uncertainty in the soil properties, structure properties, and in the SSI analyses. ASCE 4 specifies that these uncertainties can be adequately captured by conducting SSI analyses for three soil cases and using the maximum response from these three analyses for structural design. The three soil cases are based upon a lower bound soil stiffness, best estimate soil stiffness, and an upper bound soil stiffness. The best estimate soil stiffness will be established from the median soil data. Soil stiffness variability is established from statistical evaluation of data determined from geophysical measurements of the site borings as specified in ASCE 4. Lower bound, best estimate, and upper bound low strain soil stiffness properties are used as input for site response analyses to establish earthquake strain level properties for each of the three soil cases.

1.5 Structural Analysis

A. Load effects consisting of member forces, stresses, and deformations shall be determined by means of structural analyses using the loads described in Section III.1.1. For all loads except for earthquake and accidental blast, structural analysis shall be accomplished by means of a linear static analysis. For earthquake or accidental blast considerations, other structural analysis methods are used as discussed in this section.

B. For PC-3 and PC-4 structures, seismic demand (D_s as shown in Equation III-3) shall be computed in accordance with the requirements of ASCE 4. Seismic demand shall be computed using linear equivalent-static analysis, linear dynamic analysis, complex frequency response methods, or nonlinear analysis in accordance with the following discussion and ASCE 4. Regardless of the procedure followed, it is required that:

- The input to the structure be defined by the DBE response spectra or compatible time histories.
- The important frequencies of the structure be estimated, or the peak of the DBE spectrum, multiplied by an appropriate factor (as discussed below) be used as input.

- Soil-structure interaction and multi-mode effects shall be considered.
- A load path evaluation for seismic induced forces be performed. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the foundation.
- Seismic demand shall be obtained for the three orthogonal (two horizontal and one vertical) components of earthquake motion in accordance with ASCE 4. In general, the orthogonal axes shall be aligned with the principal axes of the structure.
- All vertical load path elements not part of the lateral force resisting system shall be able to safely withstand the lateral displacements induced by seismic loads on the structure.

1.5.1 Linear Seismic Analysis

A. Linear Equivalent-Static Seismic Analysis

1. An equivalent-static analysis may be used to evaluate single-point-of-attachment cantilever models with essentially uniform mass distribution, or other simple structures that can be idealized as a single degree of freedom system. For cantilever models with essentially uniform mass distribution, the equivalent-static-load base shear shall be determined by multiplying the cantilevered structure, equipment, or component masses by an acceleration equal to the peak of the input response spectrum. For these structures, the base moment shall be determined by using an acceleration equal to 1.1 times the peak of the applicable response spectrum. The resulting load shall be applied at the center of gravity of the structure.
2. For cantilevers with nonuniform mass distribution and other simple multi-degree-of-freedom structures in which the predominant or fundamental mode of the structure has curvature in one direction only (similar to a cantilever mode), the equivalent-static load shall be determined by multiplying the structure, equipment, or component masses by an acceleration equal to 1.5 times the peak acceleration of the applicable response spectrum. A smaller factor may be used, if justified.
3. Alternately, the spectral acceleration value at the fundamental frequency of the structure may be used if a modal solution has been obtained in accordance with ASCE 4. The use of the 1.1 or 1.5 factors, as defined above, are required.

B. Linear Dynamic Seismic Analysis

1. Linear dynamic analysis may be used for any structure and may be performed using either response spectrum or time history approaches. Time history approaches may use either direct integration or modal superposition methods in accordance with ASCE 4, Section 3.2.2. P-Δ effects shall be included, if significant. If inclusion of P-Δ effects results in greater than a 10% increase in the imposed moment demand on a structural member, the effects shall be included; otherwise, they may be neglected.

1.5.2 Nonlinear Seismic Analysis

A. Nonlinear Seismic Response Analysis

1. Nonlinear seismic response analysis may be required when significant nonlinear behavior is expected in some elements or when significant irregularities exist. This method requires definition of the load-deformation behavior of individual elements or the overall structural system. The nonlinear load-deformation curves used in analysis shall reflect behavior based on experimental data, which may be approximated by linear or curved segments. Nonlinear behavior shall be determined under monotonically increasing lateral deformation when nonlinear static analysis (pushover analysis) is performed. In the case of nonlinear dynamic analysis, appropriate load-deformation curves incorporating multiple reversed deformation cycles shall be used.

B. Nonlinear Static Analysis

1. Structures whose response is dominated by a single mode may be evaluated using a nonlinear equivalent-static (pushover) analysis, provided that an effective frequency and damping are used to quantify the nonlinear response. Nonlinear equivalent-static methods of analysis shall follow the guidance provided in FEMA-356 for the target displacement method or in ATC-40 for the capacity spectrum method.

C. Nonlinear Dynamic Analysis

1. Nonlinear dynamic procedures shall follow the guidance provided in ASCE 4, Section 3.2. Nonlinear dynamic analysis shall:
    1. have sufficient degrees of freedom to represent important responses of the structure. Single degree of freedom models may be used for structures whose response is dominated by a single mode.
    2. include P-Δ forces, if significant.
    3. appropriately represent both the monotonic (backbone) and cyclic behavior of nonlinear elements. Members that exhibit pinched hysteretic behavior in laboratory tests shall be represented in the analysis with elements that represent similar pinching characteristics. Mean force-deflection properties shall be used.
    4. plastic hinge lengths for frame members can be approximated by one beam depth, developed by rational analysis, or justified by comparison to test data.
2. When performing such nonlinear calculations, at least three different modified recorded accelerograms shall be used to determine potential nonlinear response. If less than five accelerograms are used, the largest response shall be used in making demand to capacity checks. If five or more accelerograms are used, the mean of the calculated responses may be used in making demand to capacity checks. If design spectrum matching is done separately for the low frequency (about 1 hz) and high frequency (about 10 hz) ranges, then at least 3 time histories are required for each frequency range.

1.5.3 Evaluation of Story Drift and P-Delta Effects

A. The load effects including forces, stresses, and deformations throughout the structure are determined from the analysis for earthquake loads as defined above. The resulting displacements must be used to evaluate story drift and to assess potential P-Delta effects. The design story drift, Δ is computed as the difference of the deflections at the top and bottom of the story under consideration. Story drift is determined from the following:

$$\Delta = \delta_x - \delta_{x-1}$$
$$\delta_x = SF \cdot \delta_{xe} \qquad \text{Eq. III-8}$$

where δ_x is the deflection of Level x in the structure and δ_{xe} is the deflection at Level x determined by the elastic analysis subjected to earthquake load. Note that for evaluation of deflections, the response deflections are scaled by SF (as defined in Section III.1.B.2) but are not reduced by the inelastic energy absorption factor, F_μ.

B. Where P-Delta effects are found to be significant, the resulting earthquake load effects (forces, stresses, and deformations) must be amplified. The significance of P-Delta effects are determined by the evaluation of a stability coefficient, θ, as follows:

$$\theta = \frac{P_x \Delta}{V_x h_{sx}} \qquad \text{Eq. III-9}$$

where P_x is the total vertical design load at and above Level x (when computing P_x, no individual load factor need exceed 1.0). V_x is seismic shear force acting between Levels x and x-1, and h_{sx} is the story height below Level x. Δ is the story drift associated with shear force V_x.

C. P-Delta effects are not considered significant when θ is equal to or less than 0.10. The structure is considered unstable if θ is greater than θ_{max} as defined below:

$$\theta_{max} = \frac{0.5}{\beta} \leq 0.25 \qquad \text{Eq. III-10}$$

where β is the ratio of shear demand to shear capacity for the story between Level x and x-1. This ratio may be conservatively taken as 1.0. When θ is greater than θ_{max}, the structures is potentially unstable and shall be redesigned. When θ is between 0.10 and θ_{max}, the story drift and member forces and moments shall be amplified by the ratio of 1.0/(1-θ).

1.5.4 Evaluation of Blast Effects

A. PC-3 and PC-4 structures may be subjected to blast effects from either designed experiment blast loads, L_{EB}, or from accidental blast loads, A_B. For designed experiment blast conditions, it is required that shielding or containment be provided by internal structures such that blast loads on the PC-3 and PC-4 building structure are minimized. As mentioned previously, the design of internal containment structures or shielding for experimental explosions is not within the scope of this chapter. However, the building could experience reaction forces from the internal containment structure or blast effects from exterior experimental explosions. Since the building structure may be subjected to these experimental blast loads repetitively, the resulting loads must be sufficiently small that progressive damage to the building does not occur. Structural response must be limited to elastic behavior. Hence, the designed experiment blast loads, L_{EB}, on the building structure are consistent with live load. Experimental blast loads are short duration, pulse loads. *Analysis for such loads may be conservatively performed by linear static analysis at the peak loading. Alternately, the dynamic nature of the load may be accounted for to obtain more realistic results.*

B. This section covers structural analysis of the building structure for accidental blast, A_B loadings. Accidental explosions may occur during handling of high explosive materials resulting in a detonation or due to a release of hydrocarbons followed by ignition resulting in a vapor cloud explosion or deflagration. In either case, the resulting loads on building structures can be of very large amplitude depending on the distance of the building from the explosion, but these loads will be of very short duration in a single pulse, on the order of a fraction of a second.

C. Because of the short duration characteristic of blast loadings, the dynamic characteristics of the building structure typically effectively reduces the blast effects, as the period of the structure is usually much greater than the duration of the load. In addition, ductile structures can undergo extensive inelastic behavior, short of collapse, during these short duration blast loads. The short duration blast pulse imparts relatively low kinetic energy that is absorbed during strain energy of structural members responding in the nonlinear range.

D. Because the amplitude of blast overpressure acting on building surfaces can be very large compared to earthquake or wind forces acting on these surfaces, it is necessary to account for yielding of building structural members in order to obtain an economical design. Hence, structural analysis for accidental blast loads is accomplished by nonlinear response history analyses. Such analysis may be accomplished by nonlinear, dynamic finite element computer programs. Alternately, there are more simple approximate methods (e.g., TM 5-1300 [22]) that fully account for both the dynamic character of the structure and the blast load; and the nonlinear behavior of the structure withstanding the blast loads.

E. The first step in the blast structural evaluation is to evaluate the loads acting on the building walls and roof in terms of blast overpressure, impulse, and duration at the building location. Building walls facing the explosion source will be subjected to increased reflected pressure. Hence, the location of potential explosion sources must be identified and accounted for in the determination of building loads. The walls not facing the explosion source and the roof will also be subjected to blast pressure loads. Techniques are available for estimating building loads in TM 5-1300 [22], DOE/TIC-11268 [32], and the ASCE report on blast resistant design in petrochemical facilities [44]. The ASCE publication "Structural Design for Physical Security, State of the Practice" [50] is a useful supplement to these design references.

F. Building or component response is then determined by nonlinear time history analyses or by simplified approximate methods per TM 5-1300 [22] or other similar references. Techniques for these analyses are found in the references cited above. Response quantities of interest include forces, moments, stresses, and displacements; as well as plastic hinge locations and support rotations and ductility levels to measure the level of response inelastic behavior.

1.6 Acceptance Criteria and Other Design Requirements

A. The basic structural design requirements are presented in Section III.1.3. This section provides the measures that demonstrate that those requirements are met. For non-seismic or non-blast loads, the members of the structure must have sufficient strength and stiffness to withstand the applied loads in accordance with ASCE 7, the IBC, or the applicable material standard. For seismic and blast loads, the structure must also have sufficient strength and stiffness to withstand the applied loads. However, for these limited energy, dynamic loads, the structure must also have sufficient energy dissipation capacity. This latter requirement necessitates a number of required design detailing measures that are necessary to achieve satisfactory structural performance. The basic provisions for assuring structural members have adequate strength and stiffness to withstand all applied loads and the additional design requirements essential to achieve satisfactory seismic or blast structural performance are presented in this section.

1.6.1 Evaluation of Total Demand

A. Following the structural analysis methods described in Section III.1.5, the element forces, moments, and/or stresses, and joint displacements are determined for each of the loads described in Section III.1.1. From the seismic analysis, the effects of seismic (earthquake-induced) forces, D_S is determined. From linear elastic analyses, D_S is reduced by F_μ to include the effects of inelastic energy absorption of the structure during transient earthquake ground motion. The computed seismic demand from nonlinear analyses included inelastic energy absorption effects and is not reduced by F_μ. From either linear or nonlinear analyses, the seismic response is multiplied by scale factor, SF, depending on the performance category (see Section III.1.2.2).

B. The next step is to combine responses from seismic and other concurrent loadings to evaluate the total demand forces, D for the applicable load combinations described in III.1.2. *For building structures, a computer analysis will normally be performed. It is advantageous to apply each non-seismic load case and the seismic load cases one at a time. Through diligent use of the load combination capabilities of the computer software, the load combinations in III.1.2. can be readily performed and the possibility of a member response being governed when one load is not active (when it counteracts the other loads) may be systematically investigated.*

1.6.2 Reinforced Concrete Structure Design Requirements

A. PC-3 and PC-4 concrete structures shall comply with the provisions of ACI 349 [20] as described below.

B. The provisions of ACI 349 [20] shall be applied for determining the construction requirements and capacity of the reinforced concrete components of PC-3 and PC-4 structures. The load equations of Section 9.2 of ACI 349 do not apply. The ultimate member loads (M_u, P_u, and V_u) shall be determined by applying the load combination equations of Section III.1.2.

C. The basic requirement of ACI 349 is that the capacity calculated using the provisions of ACI 349 [16], which is equal to ΦM_n, ΦP_n, and ΦV_n (where M_n, P_n, and V_n are the nominal capacities using ultimate strength methods and Φ are the appropriate capacity reduction factors) is greater than the limiting demand (M_u, P_u, and V_u) calculated using the load combinations in Section III.1.2.

D. Section 10.11, 10.12 and 10.13 of ACI 349 include a methodology for determining whether moments on column members should be modified due to secondary effects (P-Δ). The provisions of these sections may be applied for concrete structures in place of the methodology for considering (or not considering) P-Δ effects discussed in Section III.1.5.3.

E. Special provisions for LANL PC-3 and PC-4 reinforced concrete structures include:

- Concrete moment frames used to resist seismic forces shall be special moment frames.
- Shear walls used to resist seismic forces shall be special reinforced concrete shear walls.
- Frame components assumed not to contribute to lateral force resistance shall conform to Section 21.7 of ACI 349 such that they can resist gravity loads at lateral displacements due to earthquake response.
- Columns supporting reactions from discontinuous stiff members such as walls shall have transverse reinforcement in accordance with ASCE 7 Section A.9.9.4.2.

F. The plastic moment capacity of concrete elements under the effects of dynamic loading stress reversal is an important element of assuring ductile behavior. New concrete structures shall meet the detailing requirements (minimum and maximum reinforcement, etc.) in Chapter 21 of ACI 349 [20]. In addition the following recommended design and detailing principals should be followed:

- *When providing the required development length of the reinforcing bars, the plastic hinge size shall be considered (the development length shall be measured from the hinge end).*

- *Structural integrity of the concrete in the hinge compression zone must be assured. Providing stirrups and ties in the hinge area for beams and columns, respectively, limits the degradation in the moment capacity under repeated stress cycling.*
- *Compression reinforcing steel bars assist the concrete in the hinge area in carrying the compression loads. Adequate development, coupled with the use of ties/stirrups, as discussed above, improves their efficiency under repeated stress cycling.*
- *Positive-moment strength at the joint face shall be not less than one-half of the negative-moment strength provided at the face of the joint. Neither the negative- nor the positive-moment strength at any section along the member length shall be less than one-fourth the maximum moment strength provided at the face of either joint,*
- *Beams, columns, and walls that are part of the lateral load resisting system shall have design shear forces that approximate the maximum shear that may develop in a member. Therefore, the required shear strength for these members is related to flexural strengths of the designed member rather than to factored shear forces indicated by lateral load analysis. The design shear force is at least 125% of the shear associated with the flexural moment strength of the element.*

1.6.3 Steel Building Structures Design Requirements

A. The steel design code for structures in nuclear facilities is AISC N690 as referenced below. However, it is judged that it is preferable to design such structures by LRFD as supplemented by the AISC seismic provisions. Hence, PC-3 and PC-4 steel structures shall be designed and detailed in accordance with AISC LRFD provisions and the AISC seismic provisions. Reference documents for steel design are listed below:

- American Institute of Steel Construction (AISC), "Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities," ANSI/AISC N690-1994s1, April 15, 2002.
- American Institute of Steel Construction (AISC), "Load and Resistance Factor Design Specification for Structural Steel (LRFD)," 1999.
- American Institute of Steel Construction (AISC), "Seismic Provisions for Structural Steel Buildings," ANSI/AISC 341-02, May 21, 2002.
- American Iron and Steel Institute (AISI), "Specification for the Design of Cold-Formed Steel Structural Members," 1996, including Supplement No. 1, July 30,1999.
- ASCE 7, Section 9.8 and Appendix A9.8 (2002)
- ASCE 8, "Specification for the Design of Cold-Formed Stainless Steel Structural Members," SEI/ASCI 8-02, 2002.

B. The basic requirement for LRFD is that the capacity, which is equal to ΦM_n, ΦP_n, and ΦV_n (where M_n, P_n, and V_n are the nominal capacities using ultimate strength methods and Φ are the appropriate strength reduction factors) is greater than the limiting demand (M_u, P_u, and V_u) calculated using the load combinations in Section III.1.2.

C. Special provisions for LANL PC-3 and PC-4 steel structures include:

- Steel structures shall be designed and detailed in accordance with LRFD provisions and Part I of the AISC seismic provisions or ASCE 7 Section A.9.8.6 for light-framed cold-formed steel wall systems.

- For cold-formed steel structures shall be designed for seismic loads in accordance with the AISI specification listed above with the exception that Section A5.1.3 of that specification is revised by deleting reference to earthquake or seismic loads in the sentence permitting the 0.75 factor. The load combinations in this Chapter (Table II-5) shall be used.
- Steel deck diaphragms shall be made of materials conforming to the requirements of the AISI specification listed above or ASCE 8-90. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a registered design professional experienced in testing of cold-formed steel assemblies. Nominal strengths shall be approved by the LANL Chapter 5 POC. Design strengths shall be determined by multiplying the nominal strength by a resistance factor, ϕ equal to 0.60 for mechanically connected diaphragms and equal to 0.50 for welded diaphragms. The steel deck installation for the building, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.
- The design strength of steel cables shall be determined by the provisions of ASCE 19-96, with the following exceptions. Section 5d of the standard shall be modified by substituting $1.4(T_4)$ when T_4 is the net tension in the cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of the standard.
- Chapter C of AISC-LRFD [13] includes a methodology for determining whether moments on column members should be modified due to secondary effects (P-Δ). The provisions of this section may be applied for steel structures designed using LRFD in place of the methodology for considering P-Δ effects discussed in Section III.1.5.3.

D. Steel structures shall meet the appropriate requirements of Sections 9 through 15 of Part I of "Seismic Provisions for Structural Steel Buildings", Ref. [18]. *In addition, the following recommended design and detailing principles should also be followed to assure ductile behavior of SMRF:*

- *The design shall be based on strong column-weak beam philosophy. In such a design, seismic strength of columns at a joint is greater than the seismic strength of beams.*[8] *Experience indicates that this design feature precludes catastrophic structural failure.*
- *Yielding at beam-ends shall take place away from the joints. Using reduced beam sections (RBS) or strengthening the joints satisfies this condition.*
- *Panel zone shear capacity shall be greater than the flexural capacity of the connected beam(s) so that panel zone shear deformation effects will be small.*
- *Connection design shall assure adequate strength so that no other element will yield or fail prior to yielding at the beam end. For this purpose, doubler plates and continuity plates shall be provided as needed. (FEMA 2000).*
- When moment frames are designed using welded connections, both the design and construction of the weld must take into account the factors that affect the joint behavior.[9]

8 Experience indicates that this design feature precludes catastrophic structural failure.

9 The Northridge earthquake showed that many factors might lead to premature failure of a welded joint. These factors include: notch toughness of both base and weld material, weld quality, material properties (e.g., yield strength much higher than intended), and bottom backing bar. Guidance on welding is given in FEMA 350.

E. *The recommended detailing practices for braced frames include*:

- *Only the bracing members shall be allowed to yield under extreme loads. Columns and beams in the lateral load path shall be designed to remain elastic under seismic loads.*
- *Connections shall be detailed such that connection ultimate strength (bolts and welds) can develop greater strength than the bracing yield strength in accordance with the governing material code.*
- *Bracing slenderness ratio shall be limited to $1000/(F_y)^{1/2}$, where F_y is in ksi.*
- *Bracing members shall be designed to act both in tension and compression or tension only. The sum of the tension capacity should not exceed 70% of the total lateral load in accordance with the AISC Seismic Provisions [18]. Tension only bracing is not allowed.*
- *In general, beam to column connections are designed as simple shear connections.*
- *All elements in the lateral load path of eccentrically braced frames other than the "link," i.e., shorter segment of the beam that is intended to yield in shear, should be designed for a minimum of 125% of the beam yield strength.*
- *Elements of the compression members shall meet the width-thickness ratios of "compact" members as defined by the AISC specifications.*

1.6.4 Reinforced Masonry Structure Design Requirements

A. PC-3 and PC-4 reinforced masonry structures shall be designed and detailed in accordance with ACI 530-99 as referenced below and Appendix A9.11 of ASCE 7:

- American Concrete Institute (ACI), "Building Code Requirements for Masonry Structures," ACI 530-99/ASCE 5-99/TMS 402-99, 1999 and "Specifications for Masonry Structures," ACI 530.1-99/ASCE 6-99/TMS 602-99.

B. References to "Seismic Performance Category" in Section 1.13, and elsewhere in ACI 530-99, shall be replaced by "Seismic Design Category." Other required exceptions to ACI 530-99 are presented in ASCE 7 Appendix A9.11. Masonry structures shall meet the detailing requirements (minimum and maximum reinforcement, etc.) in Sections 1.13.4, 1.13.5, and 1.13.6 of ACI 530 [17].

1.6.5 Drift Limits

A. The story drift due to seismic response, Δ, as determined by Equation III-8 shall not exceed the allowable story drift for any story as presented below. Evaluate story drifts due to lateral forces, including both translation and torsion. For structures with significant torsional deflections, the maximum drift shall include torsional effects. It may be assumed that inelastic drifts are adequately approximated by elastic analyses (note that lateral seismic forces are not reduced by F_μ when computing story drifts). The calculated story drift do include the scale factor, SF, as shown in Equation III-8. Calculated story drifts shall not exceed 0.010 times the story height for moment frame structures. For shear wall and braced frame structures, the calculated story drift shall not exceed 0.004 times the story height. These drift limits may be exceeded when acceptable performance of both the structure and nonstructural elements can be demonstrated at greater drift.

B. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact in accordance with Section III.1.6.6.

1.6.6 Building Separation

A. Consider the relative deformations between adjacent structures to prevent potential pounding during lateral loads due to earthquake or wind. A minimum separation of S, calculated as follows, shall be maintained between adjacent structures:

$$S = 2.0\ (\delta_{x1}^{2} + \delta_{x2}^{2})^{1/2} \qquad \text{Eq. III-11}$$

where δ_{x1} and δ_{x2} are the maximum displacements along the same axis for the adjacent building structures computed in accordance with Equation III-8.

1.7 Additional Structural Design Considerations

1.7.1 Minimum Antiterrorism Structural Design Measures

A. Structural design measures on progressive collapse avoidance and on window protection presented in the DoD Minimum Antiterrorism Standards for Buildings [46] shall be considered for those buildings where there is a credible terrorist threat at the building. LANL shall specify whether these minimum antiterrorism measures are to be implemented. The following guidelines are provided to assist the project manager in making this determination[10]:

- ML-1 or ML-2 nuclear facilities
- Buildings with high occupancy (greater than 300 occupants)
- Buildings with high consequence of failure (high risk, essential mission, etc.)
- Important buildings for which potential terrorist threats are not mitigated by other security measures

B. Progressive collapse provisions from Ref. 46 are additions to the continuity and redundancy requirements for seismic design. By these provisions, columns are designed to accommodate the loss of lateral support from one floor such that columns are designed for increased unbraced length. In addition, the structure is designed such that an exterior member (beam or column) can be removed without collapse of the building. Also, all floors are to be designed with improved capacity to withstand load reversals due to explosive effects by designing them to withstand a net uplift equal to the dead load plus one half the live load. Provisions for windows include the use of laminated glass of a required minimum thickness and the design of window frames for a minimum load on the glass surface.

1.7.2 Seismic Design and Detailing Requirements

A. The design and detailing of the components of the seismic force resisting system shall comply with Section 9.5 of ASCE 7, the IBC, and applicable material standards. This section highlights some important design and detailing requirements from these other sources.

10 This list is not mean to be all inclusive, but is provided to assist the project manage in specifying minimum antiterrorism requirements.

B. All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force resisting system, and the connections shall be capable of transmitting the seismic force, F_p induced by the parts being connected. The seismic force, F_p shall be determined from the in-structure response spectrum at the location of the connection.

C. Where openings occur in shear walls, diaphragms, or other plate-type elements, reinforcement at the edges of the opening shall be designed to transfer the stresses into the plate structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement. The extension must be sufficient in length to allow dissipation or transfer of the force without exceeding the shear and tension capacity of the diaphragm or the wall.

D. Redundancy shall be considered in structural design. The design of a structure shall consider the potentially adverse effect that failure of a single member, connection, or component of the seismic force resisting system will have on the stability of the structure.

E. Collector elements shall be provided that are capable of transferring the seismic forces originating in portions of the structure to the element providing the resistance to those forces. Collector elements, splices, and their connections to resisting elements shall resist seismic loads not reduced by the inelastic energy absorption factor, F_μ.

F. Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or which are supported by the wall. The anchorage shall provide a direction connection between the walls and the roof or floor construction. Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force determined from the in-structure response spectrum at the point of attachment. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. The continuous ties required shall be in addition to diaphragm sheathing for wood diaphragms and the metal deck shall not be used as the continuous ties for metal deck diaphragms. The strength design forces of steel elements of the wall anchorage system, other than anchor bolts and reinforcing steel shall be seismic loads not reduced by the inelastic energy absorption factor, F_μ. Diaphragm to wall anchorage using embedded straps shall be attached to, hooked around or other wise anchored to the diaphragm reinforcing steel so as to effectively transfer forces to the diaphragm reinforcement. Within the wall these embedded straps shall be stand-alone anchorage that does not apply lateral loading to the vertical wall reinforcing steel.

G. Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having out-of-plane offset plan irregularities or the vertical irregularity of in-plane discontinuity in vertical lateral force resisting elements shall have the design strength to resist the maximum axial force that can develop in accordance with the seismic loads not reduced by the inelastic energy absorption factor, F_μ.

H. The deflection in the plane of a diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity under the individual loading and continue to support the prescribed loads.

1.7.3 Additional Foundation Detailing Requirements

A. In addition to detailing requirements given in ASCE 7 Section 9.7 and the IBC, detailing of PC-3 and PC-4 building foundations shall meet the following requirements[11]:

- Minimum embedment depth of foundations is 36 inches unless the foundation sets directly on welded tuff. *This provision is to prevent foundation problems due to frost heaving.*
- Interconnect all spread footing type foundations using tie beams. The tie beam shall be capable of resisting, in tension or compression, a minimum horizontal force equal to 10% of the larger column vertical load. The tie beams shall also be capable of resisting bending due to prescribed differential settlements of the interconnected footings as stipulated by the project geotechnical engineer and to eccentric positioning of columns and corresponding column loads on spread footings, simultaneously with the horizontal force.
- The design for PC-3 and PC-4 structures shall provide for a minimum horizontal and vertical differential movement between footings of an amount provided by the Chapter 5 POC depending on the site geology and seismic hazard.[12]
- The perimeter basement walls and other subterranean structural walls shall be designed for soil pressures, including potential seismic loads, (Subsection A.8) as recommended by a licensed geotechnical engineer knowledgeable of LANL soil conditions.
- Foundations shall not be designed in locations that are within 50 feet of mapped active faults

1.7.4 Preferred Structural Properties of Buildings

A. The ductility, F_{μ}, factors given in Section III.1.2 (Table III-7) imply that inelastic deformation is permissible when structures are subjected to earthquake loadings. *Inelastic deformation means damage (although limited) to the structure and, after the design basis event, the structure may need to be repaired before it can be placed in full service. Certain principles should be adhered to in detailing new buildings to assure acceptable behavior during earthquakes. Suggested guidance include:*

- *Minimize Structural Irregularities: Structure layout and detailing should minimize both horizontal and vertical irregularities.*
- *Provide a uniform strength distribution: Members of the lateral load resisting system should be designed with nearly uniform design margin for lateral loads, in order to create a system where inelastic deformations are well-distributed throughout the structure.*
- *Maximize structural redundancy: Structures with large redundancy behave better under earthquake loads. The layout of the structure should include as much redundancy as practical.*

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11 These provisions are in addition to the detailing requirements in IBC 2003 and are considered to be additional LANL specific requirements that are conservatively applied.

12 This provision is intended to account for potential differential movements of the ground surface during an earthquake at joints, bedding planes, cracks, and minor faults.

1.7.5 Acceptable and Prohibited Structural Types for PC-3 and PC-4 Buildings

A. Structural systems specifically prohibited for use in the design of the lateral force resisting system of new PC-3 and PC-4 buildings at LANL include; ordinary moment-resisting frame systems, intermediate moment-resisting frame systems, K-braced frames, plain concrete systems, precast concrete systems (which use gravity only bearing connections), unreinforced masonry systems, and wood structures.

B. When a steel manufactured building is selected for PC-3 or PC-4 service the lateral force resisting system must comply with requirements of Section III. *Note that tension only bracing is not allowed per AISC Seismic Provisions [18] and this standard.* In addition, it must be demonstrated by testing that impact by the wind-driven missiles defined in Table III-4 will not compromise the confinement integrity of the wall and roof panels. These requirements must be clearly stated in the procurement specifications and enforced during construction.

C. PC-3 and PC-4 buildings shall be either reinforced concrete, reinforced masonry, or steel buildings that are special moment resisting frames (SMRF)[13], braced frames and eccentrically braced frames (cross bracing, concentric bracing and eccentric bracing. Chevron bracing shall be used only when other alternatives are not feasible).[14]

1.7.6 Nondestructive Examination

A. Steel PC-3 and PC-4 structures shall be subject to the enhanced nondestructive examinations described in Section Q1.26 of AISC N690 [19].

1.7.7 Specification Templates (Programmatic and Facility)

A. The LANL Standards Program maintains standard specifications for vendors that provide and construct PC-1 and PC-2 building structures as well as certain nonstructural components at the LANL site. *These specifications should be edited to suit the particular project. Specifications available for editing and use include:*

Section 03100 – Concrete Formwork
Section 03200 – Concrete Reinforcement
Section 03300 – Reinforced Concrete
Section 04300 – Unit Masonry System
Section 05120 – Structural Steel
Section 05210 – Steel Joists and Joist Girders
Section 05311 – Steel Roof Deck

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13 Moment-resisting steel frames resist the lateral loads in bending. Seismic energy dissipation is through inelastic bending of members at or near the joints. Moment frames are inherently more flexible and thus their design may be controlled by drift limits. Thus, P-Δ effects may impact the behavior, especially for taller structures; and must be accounted for in design.

14 Seismic energy dissipation in braced frames is through inelastic deformation of the bracing members. Braced frames are more rigid; thus drift limits seldom control the design. Seismic energy dissipation in eccentrically braced frames is through the "link," i.e., shorter segment of the beam that is intended to yield in shear.

Section 05313 – Steel Floor Deck
Section 13125 – Pre-Engineered Metal Buildings
Section 03310 – Cast-In-Place Concrete
Section 05520 – Handrails and Railings
Section 05531 – Gratings and Floor Plates
Section 05400 – Cold Formed Metal Framing
Section 13074 – Pressure Relief Wall Panels
Section 13085 – Seismic Protection
Section 14645 – Bridge Cranes

2.0 DESIGN AND ANALYSIS REQUIREMENTS FOR PC-3 & PC-4 NONSTRUCTURAL SYSTEMS & COMPONENTS AND NON-BUILDING STRUCTURES (PROGRAMMATIC & FACILITY)

A. **Structural design of PC-3 and PC-4 nonstructural components**, such as equipment and distribution systems, as addressed in this chapter primarily involves qualification of these items for earthquake or accidental blast loads. The design of these items for other loads is handled in other chapters of the ESM. Seismic qualification of PC-3 and PC-4 nonstructural components shall be based on equivalent static or dynamic analysis or by testing. Qualification of PC-3 and PC-4 nonstructural components for accidental blast loads shall be based on analysis. The structural design of non-building structures shall provide adequate stiffness, strength, and ductility consistent with the requirements specified in Section III.1 for buildings. Non-building structures having industry standards for their structural design shall be designed in accordance with those standards with exceptions as noted in this section of the ESM.

B. **Nonstructural systems components** include architectural, mechanical, and electrical components and systems located within buildings. These systems and components are supported from the floors, walls, or roof of the building. In addition, some of these systems and components may interconnect facilities. Examples include supports for piping running between facilities and supports for cable trays in which power cables run between facilities. Examples of nonstructural components and systems include:

- Mechanical and electrical equipment,
- Architectural components,
- Glove boxes,
- Storage racks
- Platforms and walkways,
- Cranes and hoists,
- Storage tanks and pressure vessels supported within a building
- Cable trays, conduit, and piping.

C. **Non-building structures** include all self-supporting structures other than buildings. These structures are located outside buildings on their own foundation. Examples of non-building structures include:

- Storage tanks
- Stacks and chimneys
- Storage racks
- Gas transmission and distribution piping systems
- Pressure vessels
- Impoundment dikes and walls

D. This chapter provides design and seismic qualification requirements for nonstructural components, equipment, and distribution systems and non-building structures. Specific design requirements for many of these systems are primarily included in Chapter 6 – "Mechanical" and Chapter 7- "Electrical" of this ESM. In addition, specific design of these items may be in accordance with approved industry standards explicitly for the items considered.

E. For seismic design of nonstructural components, equipment, and distribution systems, there are several unique characteristics that must be considered, which are distinctly different from building seismic design. These characteristics are:

1. These systems and components may be supported by a building structure such that the input earthquake shaking is the building response to the earthquake rather than the earthquake ground shaking.
2. These systems and components may be supported at multiple points at various locations within a building or at the ground and in the building. In this case, each support point will experience a different level of shaking imposing differential deformations to the systems and components in addition to shaking response.
3. Nonstructural components, equipment, and distribution systems are typically much simpler than building structures such that they are likely to have less redundancy and inelastic energy absorption capacity.

F. Item 3 above also applies to non-building structures. These characteristics are accounted for in the design and analysis requirements of PC-3 and PC-4 nonstructural components, equipment, and distribution systems and non-building structures presented in the remainder of this chapter.

2.1 Loads and Load Combinations

2.1.1 Non-Seismic Loads on Nonstructural Components and Non-Building Structures

A. Structural loads for PC-3 and PC-4 buildings have been discussed in Section III.1.1. These same loads generally apply to systems and components. For the purpose of designing supports and anchorage, the primary loads on nonstructural components, equipment, and distribution systems are dead load and seismic loads. These systems and components typically will not be subjected to lateral soil pressure loads and designed experiment blast loads. Systems and components located within buildings will not be subjected to wind, rain, snow, or flood loads. Non-building structures located out in the open will be subjected to wind, rain, snow, or flood loads.

B. Platforms and walkways as well as cable trays may have significant live loads. In addition, cranes and hoists as well as some mechanical equipment have significant operational loads. Operational loads are included in load combinations as live loads. Systems and components can also be subjected to self-straining loads and accidental blast loads in certain situations. Storage tanks are subject to fluid loads.

C. Non-building structures shall be designed for wind loads in accordance with Section 6.5.13 of ASCE 7 [11]. This approach utilizes the velocity pressure as determined in accordance with Equation III-1 combined with net force coefficients given in Figures 6-18 through 6-22 of ASCE 7. Note that when using Equation III-1, the appropriate wind speed for the performance category as given in Table III-3 must be used (i.e., basic wind speed of 117 mph for PC-3 and of 135 mph for PC-4).

D. Self straining forces for nonstructural components are typically thermal loads. Thermal loads are considered self limiting, however, these loads shall be considered as primary loads when performing evaluation of nonstructural components and distribution systems. *Thermal loads are primarily a local effect on piping system supports, where long continuous piping segments are subject to thermal expansion that is restricted by the supports.* Piping supports shall be designed for the forces induced from the ambient temperature of the piping *(normally considered the average environmental temperature or 70^0 F)* to the maximum normal operating temperature of the piping system.

E. Nonstructural components and non-building structures could be subjected to accidental blast loads due to either inadvertent detonation of high explosives or vapor cloud explosion from the accidental release of flammable materials in a confined and congestion location. Design accidental blast loads shall be provided by LANL. Explosions result in blast overpressure, high velocity fragments, dynamic (blast wind) pressure, and ground shock effects on structures. Buildings that have large surface areas are most vulnerable to blast overpressure effects. Nonstructural components and non-building structures have less surface area and structural response is affected most greatly by wind and drag effects due to dynamic pressure and by high velocity fragments. Design of these components and structures shall consider these effects in accordance with TM 5-1300 [22] or other similar references.

2.1.2 Seismic Loads on Nonstructural Components and Non-Building Structures

A. Non-building structures shall be designed for earthquake loads given by the PC-3 and PC-4 ground response spectra presented in Section III.1.1.10. Ground response spectra anchored to peak ground acceleration of 1.0g are presented in Figure III-1 for horizontal motion and in Figure III-2 for vertical motion. These spectra are scaled by the following peak ground accelerations to obtain spectral acceleration values for design and evaluation:

- PC-3; peak ground acceleration (PGA) = 0.34g (2,500 year return period)
- PC-4; peak ground acceleration (PGA) = 0.58g (10,000 year return period)

B. The PGA values presented above are applicable to both horizontal and vertical earthquake motion. Note that the resulting DBE response spectra are for earthquake ground motion at the free ground surface (i.e., at a location not influenced by the presence of the structure).

C. Supports and anchorage for nonstructural components shall be designed for earthquake loads given by in-structure response spectra at the component support points within the supporting structure based on structural analyses and the ground response spectra presented in Section III.1.1.10 and Figures III-1 and III-2. Development of in-structure response spectra is addressed in this section along with permissible damping levels for nonstructural components.

D. Structural damping for the design and analysis of nonstructural components and non-building structures is presented in Table III-11. As discussed earlier for structures, damping values are given for three response levels, where response is measured as the average response level in the seismic load resisting elements of the system or component and their supports represented by the demand to capacity ratio, D_e/C [C = code capacity; D_e = total elastic demand ($D_e = D_{NS} + SF*D_S$), including non-seismic loads]. The relationship between D_e/C and response level is:

- $D_e/C < 0.5$ Response Level 1

- $D_e/C \approx 0.5$ to 1.0 Response Level 2
- $D_e/C > 1.0$ Response Level 3

E. Systems and components for which the design goal is to maintain structural integrity and position retention shall be evaluated at Response Level 3 damping. Systems and components for which the design goal is to continue functioning in a passive mode shall also be evaluated at Response Level 3 damping. Response Level 1 damping values shall be used for systems and components with active functional failure modes such as relay chatter or relative displacement issues that may occur at a low cabinet stress level. Massive components such as pumps and motors are expected to be lowly stressed components and Response Level 2 damping levels shall be used. Also, light welded instrument racks are expected to be lowly stressed components and Response Level 2 damping levels shall be used.

Table III-11 Damping Value for Analysis of Systems & Components

	Level 1	Level 2	Level 3
Distribution Systems			
Piping	5	5	5
Conduit	5	7	7
Cable Tray more than 50% full	5	10	15
Other Cable Trays and trays with rigid fireproofing	5	7	7
Massive, low-stressed components (pumps, motors, etc.)	2	3	3
Light welded instrument racks	2	3	3
Electrical Cabinets and other equipment	3	4	5
Liquid containing metal tanks			
Impulsive mode	2	3	4
Sloshing mode	0.5	0.5	0.5

F. Generation of in-structure response spectra at component support locations within building structures for evaluation of internal PC-3 and PC-4 systems and components shall follow the methodology in ASCE 4 [6], Section 3.4.2. Per ASCE 4, in-structure response spectra shall be developed using either:

- The time history method or
- The direct spectra to spectra method

Note that in-structure response spectra may be available for use in the design of new equipment in existing facilities. The designer is encouraged to check with the Chapter 5 POC prior to generating new in-structure response spectra for existing facilities.

G. Structural damping for the building model used to develop the in-structure response spectra are given in Table III-10 (Section III.1.4.4). The applicable level of damping to be used in the structural model for developing in-structure response spectra shall be selected so as to best match the computed response of the building structure. *It is conservative to use Response Level 1 damping for generation of in-structure response spectra.*

H. It is important to account for uncertainty in the dynamic properties of the equipment, supporting structure, and supporting media when using in-structure spectra, which typically have narrow peaks. The peak broadening or peak shifting techniques outlined in ASCE 4 [6], Section 3.4.2 shall be employed to account for this uncertainty.

I. Equipment or distribution systems that are supported at multiple locations throughout a structure could have different in-structure response spectra for each support point. In such a case, it is acceptable to use a single envelope spectrum of all locations as the input to all supports to obtain the seismic inertial loads (DOE-STD-1020 [1]). Alternately, there are analytical techniques available for using different spectra at each support location or for using different input time histories at each different support.

J. Equipment or distribution systems that are supported at multiple locations throughout a structure are subjected to both seismic inertial loads and seismic anchor motion when subjected to earthquake shaking. Both of these components of seismic loading shall be considered in the design and evaluation of multiple-supported systems and components. *The seismic anchor motion (SAM) component of seismic response is usually obtained by conventional static analysis procedures. SAM seismic response, in terms of stresses and deformation in systems and components can be very significant if the relative motions of the support points are quite different. On the other hand, if all supports of a structural system supported at two or more points have identical seismic excitation, then this SAM component of seismic response does not exist.*

K. For multiple-supported components with different seismic inputs, support displacements can be obtained either from the structural response calculations of the supporting structure or from spectral displacements determined from the in-structure response spectra. The effect of relative seismic anchor displacements shall be obtained by using the worst combination of peak displacements or by proper representation of the relative phasing characteristics associated with the different support inputs. In performing an analysis of systems with multiple supports, the response from the inertial loads shall be combined with responses obtained from the seismic anchor displacement analysis of the system by the square root of the sum of the squares as shown in Equation III-12.

$$D_S = \sqrt{D_{inertia}^2 + D_{SAM}^2} \qquad \text{Eq. III-12}$$

where D_S is the seismic response parameter of interest (i.e., the seismic demand).

2.1.3 Load Combinations

A. Load combinations for PC-3 and PC-4 building structures were presented in Section III.1.B. These load combinations also apply to the design of PC-3 and PC-4 nonstructural components and non-building structures. For PC-3 and PC-4 design and evaluation, strength design (or LRFD) methodology shall be used. Load combinations that do not include wind or seismic loads are described in Section III.1.B.1 and Table III-6. Load combinations that include seismic loads are described in Section III.1.2.2 and Equation III-3. Load combinations that include wind loads are described in Section III.1.2.3 and Table III-8. Load combinations that include accidental blast loads are described in Section III.1.24 and Equation III-6.

B. For PC-3 and PC-4 systems and components, seismic design and evaluation shall be based on dynamic analysis or testing. For seismic analysis of these systems and components, the load combination rules presented in this section shall be used. For seismic testing of systems and components, the non-seismic loads expected during normal operations shall be simulated during the test.

C. Load combinations that include seismic loads are described in Section III.1.2.2 and Equations III-3, III-4, and III-5. PC-3 and PC-4 systems and components shall also be proportioned and detailed to resist the effects of other loads in accordance with the combinations shown in Table III-6. Seismic load combinations shall be based on strength based acceptance criteria and shall consider the inelastic energy absorption factor, F_μ and scale factor, SF as shown in Equation III-3 and repeated below:

$$D = D_{NS} + \frac{SF \cdot D_S}{F_\mu} \qquad \text{Eq. III-13}$$

where:

D = the total demand acting on an element.
D_{NS} = the non-seismic demand acting on an element. Non-seismic demand shall include the mean effects of dead, live, equipment, fluid, operating loads, as well as accident loads occurring due to the earthquake.
D_S = the calculated seismic response to the DBE using an elastic analysis approach (by either equivalent static or response spectrum analysis)
F_μ = an inelastic energy absorption factor for systems and components.
SF = a scale factor based on performance category target performance goals

Note that D_S is the combination of inertial loads and seismic anchor motion in accordance with Equation III-12 for multiple supported systems where input excitation is expected to be different at each support point.

D. The same scale factors used for the seismic evaluation of the building structure shall be used for the evaluation of systems and components. These scale factors are 0.9 for PC-3 and 1.25 for PC-4 or the alternate scale factors computed from the slope of the site-specific seismic hazard curves as described in Appendix C of DOE-STD-1020.

E. For supports of nonstructural components that consist of lateral force resisting systems similar to that found in buildings, the inelastic energy absorption factor, F_μ shall be taken as F'_μ from Table III-7. For stiff components (i.e., frequency greater than the peak of the in-structure response spectrum), F_μ shall be taken as unity. For non-building structures, the inelastic energy absorption factor, F_μ shall be taken to be 1.0 unless a larger value is justified and approved by the LANL ESM Chapter 5 POC.

F. The inelastic energy absorption factor, F_μ for equipment and distribution systems that can behave in a ductile manner during seismic response are provided in Table III-12. The values in the table apply to ductile metal materials. Ductile metal materials have specified minimum yield to specified ultimate strength ratios less than 0.8 and plastic strain to rupture greater than 10 percent. If the component contains brittle material in the load path or brittle material is used that could affects its specified safety function, then F_μ values shall be taken as 1.0. The inelastic energy absorption factors, F_μ, in Table III-12 are applicable to equipment functioning in a passive mode. F_μ shall be taken as 1.0 for active equipment. *Active equipment are items that must change state and operate during an earthquake. In these cases, it is prudent to limit behavior to elastic response.*

G. DOE-STD-1020 [1] does not specify energy absorption factors, F_μ for equipment and distribution systems. The energy absorption factors, F_μ given in Table III-12 are those that are anticipated to be in future DOE seismic standards. It is always acceptable to use energy absorption factors, F_μ equal to 1.0 for all structures, systems, and components

H. When displacements need to be calculated as part of the seismic qualification by analysis, the demand given in Equation III-13 shall by based on F_μ equal to 1.0.

Table III-12 Equipment & Distribution Systems Inelastic Energy Absorption Factor, F_μ

	F_μ		F_μ
EQUIPMENT:		DISTRIBUTION SYSTEMS:	
Vessel	1.15	Butt Joined Groove Welded Pipe	1.25
Heat Exchanger	1.15	Socket Welded Pipe	1.15
Coolers	1.15	Threaded Pipe	1.00
Chillers	1.15	Conduit	1.25
Pumps	1.15	Instrument Tubing	1.25
Fans	1.15	Cable Trays	1.25
Valves	1.15	HVAC Duct	1.15
Dampers	1.15	TANKS	
Filters	1.25	Metal Vertical Liquid Storage Tanks	
Glove Boxes	1.15	Moment and Shear Capacity	1.25
Electrical Boards	1.15	Hoop Capacity	1.5
Electrical Racks	1.15	Tanks – Horizontal	1.15
Electrical Cabinets	1.15	EQUIPMENT SUPPORTS	1.25

2.2 Nonstructural Component Evaluation by Analysis and Testing

A. The design of mechanical equipment, piping, tubing and HVAC Duct work is the subject of Chapter 6 – "Mechanical" of this ESM. The design of electrical equipment cable tray and conduit is the subject of Chapter 7 – "Electrical" of this ESM. The scope of this chapter related to mechanical and electrical equipment is with regard to the structural design for seismic loads and seismic qualification requirements. Table III-13 below includes the applicable code or standard for the seismic design of these components. For a given SSC, the applicable demand (loadings other than earthquake) and capacity (allowable stress) criteria shall be as given in the Code or Standard listed in this table.

Table III-13
Applicable Codes and Standards for Seismic Design of Systems and Components

System, Structure, or Component	Applicable Code or Standard
Piping and Valves	•B31.3 Category M •ASME-QME or IEEE-344 (seismic requirements)
Tanks (0-15 psig)	•API-620 •BNL-52361 (seismic requirements)
Pressure Vessels (>15 psig)	ASME BPVC Section VIII, Division 1 or 2; Lethal Service
Pumps	•API-610, 674, 675 •ASME BPVC Section VIII, Division 1 and 2; Lethal Service •ASME-QME or IEEE-344 (seismic requirements)
Heat Exchanger	•TEMA STD, 7th Ed. •ASME BPVC Section VIII, Division 1 or 2; Lethal Service
HVAC Systems	•SMACNA Stds ASME AG-1 (seismic requirements)
Conduit and Cable Trays	•NEMA Stds •IEEE-628 (for seismic requirements)
Fire Protection Systems	•NFPA-13 (Including design to NFPA Section 4-14.4.3) •NFPA-14 •NFPA-24

B. Seismic qualification shall be based on equivalent static or dynamic analysis or by testing. Generally, permanent equipment and distribution systems shall be adequately supported and anchored. Whenever any of these methods or combination of them is chosen, it must be demonstrated that the equipment or distribution systems will be capable of performing all of their specified safety functions. It shall also be verified that the equipment and distribution systems do not interfere with the safety function of adjacent equipment or distribution systems. The seismic input excitation for equipment and distribution systems is the seismic in-structure response spectra or time history developed as described in Section III.2.1.2.

C. Distribution Systems

1. Distribution systems shall be evaluated by analysis and shall meet the code requirements listed in Table III-13. When analysis is performed, piping should be modeled to ensure mathematical representation of all significant modes of vibration. The models should represent best estimate stiffness and mass properties and boundary conditions of the piping system. *It is normally advantageous to analyze piping using a computer program specifically written for the analysis of piping. The following modeling guidance is recommended:*
    - *There should be at least four nodes between supports[15],*
    - *A node shall be specified at each change in direction[16],*
    - *Supports may be considered as rigid in the direction of restraint[17,]*
    - *Support mass need not be considered, branch piping with a moment of inertia of less than 25% of the run pipe need not be modeled,*
    - *Modal combination shall be by ASCE 4 [6] including proper consideration of closely spaced modes,*
    - *Missing mass shall be considered and combined as a single term with the other piping modes and combined by SRSS.*
2. *Guidance on the seismic analysis and design of buried piping can be found in the work of Antaki, (WRC-425 [26]), Adams, et. al., (Reference [27]) and ASCE 4 [6] Section 3.5.2. Use the guidance provided in BNL-52361 [25] for double containment piping.*

15 Many piping programs include a term that specifies the number of segments of a piping element. Therefore, it may be acceptable to model the piping from support to support provided the number of segments per element is four.

16 Most piping programs have special elements for elbows which determines the geometry based on a definition of the tangent point and radius. Included in the special element is a definition of the flexibility factor and stress intensification factor for the elbow.

17 Alternatively support stiffness may be considered, however, when it is considered it must be considered consistently for all the supports in the piping system.

D. Mechanical Equipment

1. Mechanical equipment shall be evaluated by analysis or test and shall meet the code requirements listed in Table III-13.[18] *When analysis is performed, the mathematical models should be based on structural parameters which are calculated or established by test, or a combination of these. The models should represent best estimate stiffness and mass properties and boundary conditions of the equipment. The model should be sufficiently refined to ensure mathematical representation of all significant modes of vibration and the evaluation of all pertinent failure modes. This shall entail sufficient detail to illustrate relative motion of key points, coupling and load transfer, etc. Where complex mathematical models are based solely on calculated structural parameters, the use of verification testing should be considered.*
2. Testing requirements when employed are described in Section III.2.2.2.

E. Electrical Equipment

1. Electrical equipment shall be evaluated by analysis or test (*testing being the preferred method*[19]) and shall meet the code requirements listed in Table III-13. *When analysis is performed, the mathematical models should be based on structural parameters which are calculated or established by test, or a combination of these. The models should represent best estimate stiffness and mass properties and boundary conditions of the equipment. The model should be sufficiently refined to ensure mathematical representation of all significant modes of vibration and the evaluation of all pertinent failure modes. This shall entail sufficient detail to illustrate relative motion of key points, coupling and load transfer, etc. Where complex mathematical models are based solely on calculated structural parameters, the use of verification testing should be considered.*
2. Testing requirements when employed are described in Section III.2.2.2.

F. Tanks and Heat Exchangers

1. Seismic adequacy of tanks and heat exchangers may be either demonstrated by meeting the DOE-GIP [8] requirements (which is a simplified analysis) or by a more detailed analysis. *When a more detailed analysis is performed for above ground tanks, the modeling criteria as described in Section 3.5.4 of ASCE 4 [6] may be used for the analysis. Also, additional guidance on the seismic analysis of buried and above ground storage tanks can be found in Brookhaven National Laboratory Report BNL-52361 [25].*

18 Large mechanical equipment is easier to analyze than to test.

19 It is difficult to model electrical equipment such that pertinent failure modes are considered. Therefore, testing is preferred. Note that qualification may be based on an identical item of tested equipment.

G. Anchorage

1. As described later in this part of the ESM, seismic adequacy of anchorage may be demonstrated by meeting ACI 349, Appendix B [20]. Anchorage is considered as restraint against displacement axially and in shear. It is normally not necessary to model the stiffness of the anchorage in performing analysis[20], however it is acceptable to do so either using test results for the anchorage or analytical methods considering the anchor length.

2.2.1 Seismic Qualification by Analysis

A. Seismic qualification by analysis can either be an equivalent static analysis or a dynamic analysis. In either case, it is required that:

- The input to the item shall be based on the in-structure response spectrum at support points.
- The dynamic characteristics of the system or component shall be considered in developing seismic loads or the peak of the in-structure response spectrum is used.
- Seismic anchor motion shall be considered for multiple supported systems.
- A load path evaluation for seismic induced inertial forces shall be performed. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the foundation.
- Seismic demand shall be obtained and combined for the three orthogonal components (two horizontal and one vertical) of earthquake motion using SRSS or the 100-40-40 combination as described in ASCE 4 [6]. In general, the orthogonal axes shall be aligned with the principal axes of the structure.

B. The following provides the minimum requirements for each approach.

C. Equivalent Static Analysis

1. In this type of analysis, seismic inertia loads are statically applied as external forces to a static representation of the equipment or distribution system including supports and anchorage in order to determine resultant primary (equilibrium) stresses and displacements. Equivalent static loads shall be developed from the peak of the in-structure response spectra at the location of the item considered and appropriate for the Seismic Design Category of that equipment or distribution system. Analytical procedures for equipment and distribution systems shall meet the requirements of Section 3.2.5, Equivalent Static Method of ASCE 4. For equivalent static analysis of piping systems, paragraph N-1225, simplified Dynamic Analysis of ASME B&PVC Section III, Division 1, Appendix N [7] shall be used.

20 Flexibility of base anchorage can be caused by the bending of anchorage components or equipment sheet metal. Excessive eccentricities in the load path between the equipment item and the anchor is a major cause of base anchorage flexibility. Equipment base flexibility can allow excessive equipment movement, reduce SSC natural frequency, increase dynamic response and lead to higher stresses on the anchorage. Design through the use of stiffeners and concentric location of anchors can preclude excessive flexibility.

2. Support displacement of multiple supported equipment or distribution systems due to earthquake motion, termed Seismic Anchor Motions (SAM) as discussed in Section III.2.1.2 shall be considered as required in Section 3.2.5 of ASCE Std. 4 for equipment and distribution systems and paragraph N-1225 of ASME B&PVC Section III, Division 1, Appendix N for piping systems.
3. *It is acceptable for simple systems (i.e. electrical cabinet, etc.) to apply the seismic horizontal load at the c.g. of the item. For more complicated systems (i.e. distribution systems) distribute the lateral force in proportion to the mass distribution of the system or component.*
4. Free standing mechanical and electrical equipment[21] such as Pumps, Tanks, Motor Control Centers, Heat Exchanges, etc., non-building structures such as internal frame systems, steel storage racks, glove boxes, etc. and distribution systems[22] may be analyzed using an equivalent static analysis method. The applied seismic load for components with a fundamental frequency less than 33 Hz may be taken and the peak of the applicable in-structure response spectra. When the natural frequency of the component is estimated, the applied seismic load may be based on the spectral acceleration at the estimated frequency.

D. Dynamic Analysis

1. For dynamic analysis, seismic loads and resultant inertia and seismic anchor motion stresses in equipment and distribution systems shall be based on the earthquake dynamic input, i.e., in-structure response spectra or time histories, appropriate for the Performance Category for the equipment or distribution system as presented in Section III.2.A.2. In addition, the specified dynamic analysis procedures from the following documents shall be used as appropriate for the type of component:
   - Appendix N of Section III of the ASME Boiler and Pressure Vessel Code
   - ASCE Standard 4
   - IEEE 344
   - ASME QME-1

E. Total Demand for Qualification by Analysis

1. The total demand, D, on equipment and distribution systems, qualified by analysis, is given by Equation III-13.

F. Capacity Using Qualification by Analysis

1. The capacity, C, of equipment and distribution systems when seismic qualification is based on analysis methods may be defined by industry standards or be stress acceptance criteria.

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21 Equipment includes, but is not limited to, boiler, chiller, heat exchanger, pump, air-handling unit, cooling tower, control panel, motor, switch gear, transformer and life-safety equipment.

22 It includes major conduit, ducting and piping serving such machinery and equipment and fire sprinkler systems.

- **Capacity Defined by Industry Standards:** Capacity shall be based on the specified stress or load limits specified in the appropriate code or standard for the PC-3 and PC-4 equipment or distribution systems being qualified. If the capacity limits of ASME Nuclear Codes or Standards are used to define capacity, then all ASME Nuclear Code Construction Requirements shall be met except nuclear code stamping.
- **Stress Acceptance Criteria for Capacity:** Where ASME B&PVC Section III is specified, the ASME B&PVC Section III Service Level D is allowed for Limit States A and B for passive components only. For active components ASME B&PVC Section III Service Level shall be justified to assure, with high confidence, the performance of the required safety functions. Limiting the calculated stresses to be less than the yield stress of the material will satisfy this requirement for active components. Stress limits shall be limited to be less than yield in components, supports, and anchorage for active components whose design is governed by AISC.

G. Acceptance Criteria and Documentation for Qualification by Analysis

1. Acceptance criteria for qualification by analysis shall clearly demonstrate the following:

$$D \leq C \qquad \text{Eq. III-14}$$

2. Documentation shall meet the requirements of the specific code or standard, and, as a minimum, the documentation requirements for analysis as specified in IEEE 344 shall be met.

2.2.2 Seismic Qualification by Testing

A. Seismic qualification by testing can be either a proof test or a fragility test. A seismic proof test is testing where the seismic test input, usually defined as a Test Response Spectra (TRS), exceeds the specific required seismic input, usually defined as a Required Response Spectra (RRS). Seismic Fragility testing is usually performed in steps of increased levels of seismic input until a failure or malfunction occurs or the test limits of the test table are reached. In any case, the test specimen shall be mounted on the test table, i.e., shake table, simulating as close as possible the mounting in the nuclear facility. Any other non-seismic loads that exist concurrently with the seismic loading and that could affect its safety function shall be accounted for in the test program. Seismic qualification by testing shall meet the requirements of IEEE 344 or ASME QME-1 depending on the type of item being qualified. In harsh environments, where the equipment safety function can be affected, seismic testing is only one portion of the qualification program as defined in IEEE 323. Testing of in line components such as valve actuators shall meet IEEE 328.

1. The demand for qualification by test is the following:

$$D = 1.0\ D_{ns} + 1.4\ SF\ D_s \qquad \text{Eq. III-15}$$

where:

D = the total demand on the component

D_{ns} = any additional loads that are required during the test, e.g., pressure, voltage. Also, D_{ns} could be any preaging required before the seismic test. Typically D_{ns} is zero other than the dead weight of components.

D_s = the seismic demand from an elastic seismic analysis of the supporting structure with the support structure $F\mu$ set to 1.0. Typically, D_s is defined as the in-structure response spectra of the support location of the component.

The factor 1.4 is the equipment capacity factor for qualification by test that provides the margin to obtain the required confidence level of performance.

Scale factor, SF, is 0.9 for PC-3 and 1.25 for PC-4.

B. There is no inelastic energy absorption factor, $F\mu$, since the required component performance is indirectly specified with the test criteria (e.g., functional requirements, structural integrity), which defines the component's acceptance criteria.

C. Capacity, C, by testing is defined as the applied test motion at the support location for which the component demonstrated its required function. This is typically defined as the Test Response Spectrum (TRS). The specific requirements concerning test capacity shall satisfy the latest revision of IEEE 344 or ASME QME-1 as appropriate for the component being qualified.

D. Acceptance criteria for qualification by test are defined by Eq. III-14. Compliance with Eq. III-14 at a particular frequency or range of frequencies shall satisfy IEEE 344 and ASME QME-1, as appropriate to the type of component and qualification method. Functional requirements of the component must also be addressed in the qualification procedure.

E. Documentation shall meet the requirements of the standard, code, or procedure used. The documentation shall clearly demonstrate that capacity is greater than demand for the specified functional requirements.

2.3 Nonstructural Component Design

2.3.1 General

A. This section primarily covers the design of supports and anchorage of nonstructural systems and components. This includes complete requirements for the seismic design of those supports and anchorage. In addition, the section does provide some information and requirements for the seismic design of the systems and components.

B. This section describes the design requirements (capacity) for the evaluation of PC-3 and PC-4 nonstructural components and distribution systems. Mechanical and electrical equipment may be qualified by either analysis, experience data, or testing. *PC-3 and PC-4 mechanical and electrical equipment and distribution systems have safety and/or radiological consequences with regard to their failure. Seismic qualification requirements for these items are a safety issue. These structures, systems, and components may be seismically qualified using analysis or test methods as described in Section III.2.2, using the applicable code listed in Table III-13. Distribution systems and tanks and heat exchangers are typically qualified by analysis.*

C. PC-3 and PC-4 nonstructural components shall be designed in the manner described in the previous section to assure that the component capacity is sufficiently large to safely withstand imposed demand from the various loads acting on the systems or components. Additional design requirements are presented in this section are intended to assure that all architectural, mechanical, electrical, and other nonstructural components in structures actually have that capacity (stresses or deformations). The design and evaluation of support structures and architectural components and equipment shall consider their flexibility as well as their strength. The functional and physical interrelationship of components and their effect on each other shall be designed so that the failure of a PC-3 or PC-4 architectural, mechanical, or electrical component shall not cause the failure of a nearby PC-3 or PC-4 mechanical or electrical component.

D. Components shall be attached such that component forces due to combined loads on the component are transferred to the structure. Component attachments for seismic loads shall be bolted, welded, or otherwise positively fastened without consideration of the frictional resistance produced by gravity. A continuous load path of sufficient strength and stiffness between the component and supporting structure shall be provided. Local elements of the structure shall be designed and constructed for the component forces where they control the design of the elements or their connections. The DBD shall provide sufficient information relating to component attachment to verify compliance with these requirements.

2.3.2 Component Anchorage

A. Components shall be anchored in accordance with the following provisions:

- The force in the connected part shall be determined based seismic analyses as described in Section III.2.2.1.
- Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:
  - The design strength of the connected part
  - The force in the connected part determined from elastic seismic analysis (i.e., $F_\mu = 1.0$)
  - The maximum force that can be transferred to the connected part by the component structural system
- Determination of forces in anchors shall take into account the expected conditions of installation including eccentricities and prying effects.
- Determination of force distribution of multiple anchors at one location shall take into account the stiffness of the connected system and its ability to redistribute loads to other anchors in the group beyond yield.
- Expansion anchors shall not be used where operational vibration loads are present.[23]
- Steel base plates for supports resisting significant overturning moment in addition to axial and shear forces shall have no fewer than four anchor bolts. This requirement will assure that overturning moments are not resisted solely by prying action between the anchor bolts and the base plate.
- Powder driven fasteners shall not be used for tension load applications unless approved by the LANL Chapter 5 POC.

23 Expansion anchors are unreliable in applications with operational vibration loads such as anchors for pumps and therefore, these applications should be avoided.

- The design strength of anchors in concrete shall be determined in accordance with the provisions of ACI 349, Appendix B [16].
  - Anchors shall be designed to have sufficient embedment such that the capacity of the anchorage is controlled by the anchor bolt material and not the concrete or masonry to which it is attached.[24]
- The capacity of the welded connection shall be determined using the requirements of AISC-LRFD [13].
- Anchorage to masonry structural elements may be designed by either ASD or SD measures.[25] For ASD anchorage to masonry design:
  - Allowable loads for anchor bolts such as expansion anchors, toggle bolts, sleeve anchors, etc. (all anchors not solidly grouted into masonry) shall be determined by test as described in Section 2.1.4.1 of ACI 530 [17].[26]
  - Allowable loads for plate, headed, and bent bar anchor bolts shall be determined using the criteria described in Section 2.1.4.2 of ACI 530 [17], or by test as described in Section 2.1.4.1 of ACI 530 [17].
  - Shear/tension interaction shall be linear as described in Section 2.1.4.2.4 of ACI 530 [17].
- For SD anchorage to masonry design:
  - There is no SD criteria for determining allowable loads for anchor bolts such as expansion anchors, toggle bolts, sleeve anchors, etc. (all anchors not solidly grouted into masonry). Therefore, these anchorages shall be designed as described above for ASD.
  - Allowable loads for plate, headed, and bent bar anchor bolts shall be determined using the criteria described in Section 3.1.6 of ACI 530 [17], using the understrength, Φ, factors described in Section 3.1.4.4 of ACI 530 [17].
  - Shear/tension interaction shall be linear as described in Section 3.1.6.4 of ACI 530 [17].

B. *The most reliable anchorage will be achieved by properly installed cast-in-place bolts or headed studs, undercut type expansion anchors, or welding. Other expansion anchors are less desirable than cast-in-place, undercut, or welded anchorage for vibratory environments (i.e. support of rotating machinery), for very heavy equipment, or for sustained tension supports. Epoxy grouted anchorage is considered to be the least reliable in elevated temperature or radiation environments and shall not be used in such applications without documented qualification following ACI 349, Appendix B, Section B.12 [20].*

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24 This requirement provides for ductile anchorage that is good practice for earthquake design.

25 Use of ASD is conservative since load combinations for PC-3 and PC-4 as described herein are based on strength design.

26 Tests are performed per ASTM E 448 except that a minimum or five tests shall be conducted and the allowable load for meeting the ASD design loads may not be greater than the average of the tests divided by a factor of safety or five.

2.3.3 Component Design Provisions

A. Architectural components shall be designed in accordance with the Seismic Design Category D provisions of Section 9.6.2 of ASCE 7 [11]. Mechanical and electrical components shall be designed in accordance with the Seismic Design Category D provisions of Section 9.6.3 of ASCE 7 [11]. In both cases, this design is based on the PC-3 or PC-4 in-structure response spectrum at the component support point.

B. Architectural component design provisions in ASCE 7 Section 9.6.2 address exterior nonstructural wall elements, interior nonstructural walls and partitions, parapets, chimneys and stacks, veneer, penthouses, suspended ceilings, storage cabinets and racks, laboratory equipment, access floors, appendages and ornamentation, and glass in glazed curtain walls, glazed storefronts, and glazed partitions. Important design concepts for architectural components are to assure that these components are adequately fastened to the structure and to assure that, when fastened, the components can accommodate story drifts of the supporting structure due to earthquake ground shaking.

C. Mechanical and electrical component design provisions in Table III-13 may be supplemented by ASCE 7 Section 9.6.3 that address boilers and furnaces, pressure vessels on skirts, manufacturing and process machinery, conveyors, piping systems, HVAC systems, elevator and escalator components, trussed towers, electrical distribution systems, and equipment, and lighting fixtures. Mechanical and electrical component supports and attachments include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, snubbers, and tethers as well as elements forged or cast as a part of the mechanical or electrical component. The anchorage of mechanical and electrical components shall have sufficient strength and stiffness to allow the component to perform its intended safety function. In addition, distribution systems at the interface of adjacent structures or portions of the same structure that may move independently shall be provided with adequate flexibility to accommodate the anticipated differential movements between the portions that move independently.

D. Fire protection sprinkler systems designed and constructed in accordance with NFPA 13, *Standard for the Installation of Sprinkler Systems,* must also be shown to meet the seismic force and seismic relative displacement requirements of this section. PC-3 and PC-4 mechanical and electrical components that are required to remain operable after the design earthquake shall demonstrate operability by shake table testing or experience data[27]. Alternately, such components may demonstrate operability by rigorous stress and deformation analysis. Such components shall meet the inspection and certification requirements of Section A.9.3.4.5 of ASCE 7 [11].

27 This is an exception to DOE-STD-1020. It is judged that experience data provide an alternate, cost-effective means to qualify the operability of like-equipment subjected to earthquakes. This approach has been successfully implemented at DOE sites for a number of years using Ref. [8]. Revisions to IEEE 344 [10] and ASME QME-1 [9] to incorporate experienced based methods for new design are currently being implemented.

E. The following additional restrictions apply to the evaluation of ceilings and access floors:

- Ceiling weight shall include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the seismic force, use a ceiling weight of not less than 4 pounds per square foot.
- Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed provided the walls are not over 50 feet apart.
- W_p (Component Operating Weight) for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10 psf partition load allowance.
- Independently support light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings from the structure above.[28]

F. Cranes and hosts are to be designed to the criteria specified in AISC-ASD [12] and AISC-LRFD [13] for the load combinations given is Section III.2.1.3. The live load should include the maximum hoist load including the standard impact factors in AISC. It is also required that the crane or hoist be evaluated for earthquake when loaded with the design lift load.[29] Cranes should be designed to meet NUREG-0554, "Single-Failure-Proof Cranes for Nuclear Power Plants." LANL has developed a standard specification for vendors that provide bridge cranes at the LANL site. S*pecification, Section 14645 – Bridge Cranes should be edited to suit the particular project.*

2.3.4 Seismic Interaction

A. Potential seismic interactions between adjacent SSCs, as appropriate shall be evaluated. This evaluation may entail estimating or calculating displacements and providing sufficient clearance between the potentially interacting components or may be designed using the guidance contained in Chapter 7 of the DOE-GIP [8]. When PC-3 SSC are adjacent to PC-4 and potentially could have an adverse seismic interaction the PC-3 SSC shall be evaluated to the higher PC-4 earthquake input level. The goals for components with potential for seismic interaction are maintaining structural integrity and position retention.

2.4 Non-Building Structure Design

A. PC-3 and PC-4 non-building structures, including supports and anchorage, shall generally be designed in accordance with provisions in Section III.1 of the ESM for building structures. All non-building structures shall be designed and constructed to resist the loads listed in Section III.1.1. The design of non-building structures shall provide adequate stiffness, strength, and ductility consistent with the requirements specified in Section III.1.0 for buildings or with industry standards for the particular non-building structure considered.

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28 This is good practice to prevent the falling of these items during an earthquake on personnel working below.

29 It is necessary to design PC-3 and PC-4 cranes or hoists for earthquake loads while loaded because of the safety consequences of failure PC-3 and PC-4 SSCs. Since these cranes and hoists have lifts that could potentially interact with PC-3 and PC-4 SSCs, cranes should be single-failure-proof.

B. Loads and load combinations as presented in Sections III.1.1.0 and III.1.2.0, respectively shall be considered in the design of non-building structures. Seismic demand shall be obtained for Equation III-3 and repeated below:

$$D = D_{NS} + \frac{SF \cdot D_S}{F_\mu} \qquad \text{Eq. III-16}$$

where:

- D = the total demand acting on an element.
- D_{NS} = the non-seismic demand acting on an element. Non-seismic demand shall include the mean effects of dead, live, equipment, fluid, snow, and at-rest lateral soil loads as well as accident loads occurring due to the earthquake.
- D_S = the calculated seismic response to the DBE using an elastic analysis approach (by either response spectrum or time history analysis).
- F_μ = an inelastic energy absorption factor for structural elements.
- SF = a scale factor based on performance category target performance goals equal to 0.9 for PC-3 and 1.25 for PC-4

C. Seismic input motion is given by the ground response spectra presented in Figure III-1 for horizontal motion and Figure III-2 for vertical motion. These spectra must be scaled at all frequencies by the PGA at the mean annual exceedance probability specified for the performance category considered. These PGA values are:

- PC-3; peak ground acceleration (PGA) = 0.34g (2,500 year return period)
- PC-4; peak ground acceleration (PGA) = 0.58g (10,000 year return period)

D. The inelastic energy absorption factor, F_μ, is a reduction factor used to reduce seismic demand to account for limited inelastic behavior. For non-building structures that are similar to buildings, the inelastic energy absorption factor F_μ' is a function of the structural system as specified in Table III-7 (Section III.1.2.2). For other non-building structures such as tanks and pressure vessels that are not located within buildings, the inelastic energy absorption factor, F_μ, is given in Table III-12 (Section III.2.1.3). Note that for stiff non-building structures (frequency greater than 8 hz), Equation III-5 shall be applied as discussed in Section III.1.2.2. In situations where LANL, by specification, has limited the structure stress levels to elastic limits, F_μ shall be taken as 1.0.

E. Non-building structures that are similar to buildings shall be designed in accordance with ACI-349 for reinforced concrete construction and with AISC-LRFD for structural steel construction. Other non-building structures having industry standards for their structural design shall be designed in accordance with those standards. Industry standards for pressure vessels and tanks are listed in Table III-13. Industry standards for several other non-building structures are listed in Table 9.14.3 of ASCE 7 [11].

F. At a minimum, PC-3 and PC-4 non-building structures shall follow the seismic design provisions of Section 9.14 of ASCE 7 [11] considering the structures to be in Seismic Design Category D. Specific design requirements for non-building structures are provided in ASCE 7 Sections 9.14.6 and 9.14.7. These design provisions shall be used in combination with the PC-3 or PC-4 earthquake input as presented in Section III.1.1.10 and above, as well as the applicable loads and load combinations as stipulated in Section III.1.1.0, and III.1.2.1 and 2.2.

G. Section 9.14.6 considers non-building structures that are similar to buildings. These structures include:

- Pipe racks
- Steel storage racks
- Electrical power generating facilities
- Structural towers for tanks and vessels
- Piers and wharves (not applicable to LANL)

H. Section 9.14.7 considers non-building structures that are not similar to buildings. These structures include:

- Earth retaining structures
- Tanks and vessels
- Stacks and chimneys
- Amusement structures (not applicable to LANL)
- Special hydraulic structures (e.g., separation walls, baffle walls, weirs, etc.)
- Secondary containment systems (e.g., impoundment dikes and walls)
- Telecommunication towers

I. Modeling, analysis, and design of non-building structures shall follow the provisions for buildings as presented in Sections III.1.3, III.1.4, III.1.5, and III.1.6, with exceptions as noted in this section.

J. Non-building structures can be subjected to wind and accidental blast loads. Design for these effects shall be in accordance with the appropriate provisions from ASCE 7 [11] for wind and from TM-1300 [22] for blast.